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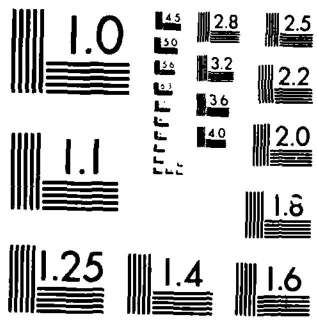


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# NCEL Technical Note

December 1985
By James A. Bailard
Sponsored By Director of
Navy Laboratories



# Simple Models for Surfzone Sediment Transport

*ABSTRACT* Energetics-based longshore and on-offshore transport models have been developed for the surfzone. The longshore transport model describes the distribution of the longshore transport rate across the surfzone. Spatial integration of this distribution produces an equation similar in form to the wave power equation but with a wave power coefficient which is a function of the breaker angle, the beach slope, and the ratio of the orbital velocity magnitude at the breakpoint divided by the fall velocity of the sediment. The on-offshore transport model describes the magnitude and direction of the spatially averaged on-offshore sediment transport rate in the surfzone. These parameters were found to be functions of the incident wave height and the mean beach slope. Both the longshore transport and the on-offshore transport models have been incorporated into a simple two-line shoreline evolution model.



NAVAL CIVIL ENGINEERING LABORATORY PORT HUENEME, CALIFORNIA 93043

## METRIC CONVERSION FACTORS

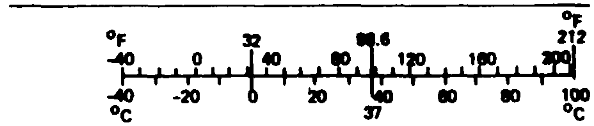
### Approximate Conversions to Metric Measures

| Symbol | When You Know | Multiply by | To Find | Symbol |
|---|---|---|---|---|
| | | **LENGTH** | | |
| in | inches | *2.5 | centimeters | cm |
| ft | feet | 30 | centimeters | cm |
| yd | yards | 0.9 | meters | m |
| mi | miles | 1.6 | kilometers | km |
| | | **AREA** | | |
| $in^2$ | square inches | 6.5 | square centimeters | $cm^2$ |
| $ft^2$ | square feet | 0.09 | square meters | $m^2$ |
| $yd^2$ | square yards | 0.8 | square meters | $m^2$ |
| $mi^2$ | square miles | 2.6 | square kilometers | $km^2$ |
| | acres | 0.4 | hectares | ha |
| | | **MASS (weight)** | | |
| oz | ounces | 28 | grams | g |
| lb | pounds | 0.45 | kilograms | kg |
| | short tons (2,000 lb) | 0.9 | tonnes | t |
| | | **VOLUME** | | |
| tsp | teaspoons | 5 | milliliters | ml |
| Tbsp | tablespoons | 15 | milliliters | ml |
| fl oz | fluid ounces | 30 | milliliters | ml |
| c | cups | 0.24 | liters | l |
| pt | pints | 0.47 | liters | l |
| qt | quarts | 0.95 | liters | l |
| gal | gallons | 3.8 | liters | l |
| $ft^3$ | cubic feet | 0.03 | cubic meters | $m^3$ |
| $yd^3$ | cubic yards | 0.76 | cubic meters | $m^3$ |
| | | **TEMPERATURE (exact)** | | |
| °F | Fahrenheit temperature | 5/9 (after subtracting 32) | Celsius temperature | °C |

*1 in = 2.54 (exactly). For other exact conversions and more detailed tables, see NBS Misc. Publ. 286, Units of Weights and Measures, Price $2.25, SD Catalog No. C13.10:286.


### Approximate Conversions from Metric Measures

| Symbol | When You Know | Multiply by | To Find | Symbol |
|---|---|---|---|---|
| | | **LENGTH** | | |
| mm | millimeters | 0.04 | inches | in |
| cm | centimeters | 0.4 | inches | in |
| m | meters | 3.3 | feet | ft |
| m | meters | 1.1 | yards | yd |
| km | kilometers | 0.6 | miles | mi |
| | | **AREA** | | |
| $cm^2$ | square centimeters | 0.16 | square inches | $in^2$ |
| $m^2$ | square meters | 1.2 | square yards | $yd^2$ |
| $km^2$ | square kilometers | 0.4 | square miles | $mi^2$ |
| ha | hectares (10,000 $m^2$) | 2.5 | acres | |
| | | **MASS (weight)** | | |
| g | grams | 0.035 | ounces | oz |
| kg | kilograms | 2.2 | pounds | lb |
| t | tonnes (1,000 kg) | 1.1 | short tons | |
| | | **VOLUME** | | |
| ml | milliliters | 0.03 | fluid ounces | fl oz |
| l | liters | 2.1 | pints | pt |
| l | liters | 1.06 | quarts | qt |
| l | liters | 0.26 | gallons | gal |
| $m^3$ | cubic meters | 35 | cubic feet | $ft^3$ |
| $m^3$ | cubic meters | 1.3 | cubic yards | $yd^3$ |
| | | **TEMPERATURE (exact)** | | |
| °C | Celsius temperature | 9/5 (then add 32) | Fahrenheit temperature | °F |



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Energetics-based longshore and on-offshore transport models have been developed for the surfzone. The longshore transport model describes the distribution of the longshore transport rate across the surfzone. Spatial integration of this distribution produces an equation similar in form to the wave power equation but with a wave power coefficient which is a function of the breaker angle, the beach slope, and the ratio of the orbital velocity


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CONTENTS

## INTRODUCTION

Quantitative models for nearshore sediment transport processes are necessary for understanding and predicting nearshore bathymetry changes. The latter include both beach profile changes and changes in the planform shape of the beach. Changes in the beach profile are most often associated with variations in the incident wave conditions, while planform changes are associated with an interruption of the littoral drift (e.g., by a groin or harbor jetty).

Beach profile changes have time scales which range from a few days when associated with a single storm, to several months when associated with seasonal wave changes. Fluctuations in the beach profile are primarily the result of on-offshore sediment transport processes and take place more or less along the entire coastline.

Beach planform changes, on the other hand, have a time scale which varies with distance from the littoral barrier. For example, in the case of a single groin the initial planform changes are restricted to the immediate vicinity of the groin. Over time, the effects of the groin are felt at increasing distances downcoast. Depending on the overall size of the groin and the rate of longshore sediment transport, the initial effects are evident within a few weeks while the cumulative downcoast effects sometimes take several years to develop.

The most widely accepted surfzone sediment transport models are simple, empirically based expressions which can be readily applied to many common coastal engineering problems. Unfortunately, the simplicity of these models and their lack of precise physical basis can lead to erroneous results when they lack the flexibility and degree of detail needed for a number of applications. More sophisticated models have been proposed, however, these models are so complex that they have had only limited application. Moreover, nearly all existing longshore and on-offshore sediment transport models lack a unity in their development, with the two transports treated as distinct processes instead of different components of the same process.

The objective of the present study was to develop a series of analytical and numerical models to predict the longshore and on-offshore sediment transport rates as well as the evolution of nearshore bathymetry. The longshore and on-offshore sediment transport model developments were to share a common energetics basis while the bathymetry evolution model was to be based on existing line model methodology. Although the models were to incorporate as much of the physics of the sediment transport processes as possible, the resulting equations were to be simple enough to be easily applied by practicing coastal engineers. Funding for this study and earlier supporting work was provided by the Independent Research Program of the Naval Civil Engineering Laboratory.

This report consists of the following sections. First, existing models for longshore transport, on-offshore transport, and nearshore bathymetry evolution are reviewed. Next, a generalized form of Bagnold's energetics-based sediment transport model for streams is reviewed, and then applied to idealized surfzone conditions. The resulting equations for the local longshore and on-offshore sediment transport rat are expressed as functions of several local wave, current, and sediment parameters. These local equations are simplifed using measured surfzone velocity field data, and combined with existing surfzone wave and current models to predict the spatially integrated longshore and on-offshore sediment transport rates as a function of incident wave parameters. Finally, both the longshore and the on-offshore transport models are combined with existing line-model methodology to numerically predict the evolution of nearshore bathymetry as a function of time.

## EXISTING SURFZONE SEDIMENT TRANSPORT MODELS

### Longshore Transport Models

Existing longshore transport models fall into two general categories, those that predict the spatially integrated longshore transport rate, and those that predict the distribution of the local longshore transport rate. The most widely accepted longshore transport model is the wave power equation, alternatively called the Scripps equation or the CERC equation (Inman and Bagnold, 1963; U.S. Army CERC, 1977). This equation linearly relates the spatially integrated immersed weight transport rate, $I_\ell$, to what has been termed the longshore component of wave energy flux, $P_\ell$, i.e.,

$$I_\ell = K P_\ell \tag{1}$$

where the wave power coefficient, K, is generally assumed to be a constant equal to 0.77 (Komar and Inman, 1970). The factor $P_\ell$ can be expressed as

$$P_\ell = (E\ Cn)_b \sin \alpha_b \cos \alpha_b \tag{2}$$

where: E = energy of the incident waves

Cn = wave group velocity

$\alpha$ = incident wave angle

$_b$ = value at breaking

The volumetric transport rate, $Q_\ell$, is related to the immersed weight transport rate by

$$Q_\ell = \frac{I_\ell}{\rho\ g\ (S_s - 1)\ N_o} \tag{3}$$

where: $\rho$ = fluid density

g = gravity

$S_s$ = specific gravity of the sediment relative to the fluid

$N_o$ = at rest volume concentration of sediment on the beach face ($N_o \approx 0.63$)

Although Komar and Inman (1970), Dean et al. (1983), Kraus et. al. (1983) and others have shown that Equation 1 does an adequate job of predicting the overall trend of longshore transport rate field data, it significantly overpredicts longshore transport rate for laboratory data (see Figure 1). Moreover, each of the above mentioned field studies suggests a somewhat different value for the wave power coefficient, K.

A possible explanation for the inability of Equation 1 to predict both laboratory and field measurements of longshore transport rates using a single value of K is that sediment transport in the laboratory may be mostly bedload while transport in the field may be mostly suspended load.

The question of the relative importance of bedload versus suspended load transport is a subject of open debate. A number of investigators (e.g., Komar, 1976) assert that most of the transport in the field is by bedload. In truth, this question may be one of semantics, for while bedload is precisely defined as all sediment which is supported in transport by grain to grain collisions, in practice bedload has often been defined as all sediment transported within 10 cm of the bottom. Obviously this latter definition includes sediments transported both as bedload and as suspended load.

The wave power equation predicts the spatially integrated longshore transport rate. A number of longshore transport models have been developed to predict the distribution of the longshore sediment transport across the surfzone. Nearly all of these models utilize an adaptation of a stream-based sediment transport model. For example, the models by Ostendorf and Madsen (197)9 and Swart (1976) utilize traction-based models, while the models by Komar (1977), Thornton (1973), and Bailard (1981) utilize energetics-based models. In order to predict the distribution of the longshore transport rate across the surfzone, these models incorporate various methods for predicting wave-induced velocities and the longshore current. Because of this added complexity, most of the models require numerical solution and have not enjoyed wide spread application.

## On-offshore Sediment Transport Models

Unlike longshore transport models, quantitative models for predicting the on-offshore sediment transport rate on a beach are scarce. One difficulty is that the time-averaged on-offshore sediment transport rate represents a small difference between the two large instantaneous onshore and offshore transports induced by each wave. As a result, secondary features in the flow field, such as wave velocity asymmetries and weak on-offshore currents, have a far greater effect on the on-offshore sediment transport rate than on the longshore sediment transport rate.

Conceptually, the on-offshore sediment transport in the surfzone can be viewed as a balance between the downslope component of gravity, the asymmetry of the orbital velocity field, and the time-averaged offshore current. Although the above factors (and others) have been recognized for some time, the complexity of the fluid/sediment motions in the surfzone have forced many investigators to seek more qualitative relationships between the incident wave properties and the direction of on-offshore sediment transport. Examples of this approach include the wave height models by Saville (1957), Aubrey (1978), and Short (1978), and the wave steepness models by Dean (1973) and Hattori and Kawamoto (1981).

A few quantitative models have been proposed including those by Swart (1974), Bowen (1980) and Bailard (1981). The latter two models are based on the energetics concepts of Bagnold (1963, 1966) which are discussed in the next section. Swart's on-offshore transport model is based on an extension of an equation proposed by Bakker (1968) and Bakker et al. (1970). This equation relates the on-offshore sediment transport rate per unit width of beach, $q_x$, to the distance between two depth contours, $x_1$ and $x_2$, as follows

$$q_x = \text{Coff}\,(x_i - x_2 + w) \qquad (4)$$

where: Coff = rate coefficient assumed constant for a specific set of boundary conditions

w = equilibrium value of $(x_2 - x_1)$

In Bakker's original formulation (used in a line model development), Coff and w were assumed to be constants, however, in Swart's model both of these parameters are allowed to vary as complex empirical functions of the incident wave conditions, the local depth, the local orbital velocity magnitude, and the sediment size. The resulting model can be used to predict the change in the beach profile as a function of time, as well as the shape of the equilibrium beach profile. The complexity of Swart's model, however, has prevented its widespread application.

Line Models

Line models, which represent the nearshore bathymetry by one or more depth contour lines, are a simple method of predicting nearshore bathymetry changes as a function of time. In a one-line model, the entire beach is represented by the shoreline (MSL) contour (see Figure 2). The beach profile is assumed constant, on-offshore sand movements are neglected, and all shoreline changes are associated with the divergence of the longshore transport rate.

Pelnard Considere (1956) developed the first one-line model by combining a linearized longshore transport equation with the continuity equation for sand. The resulting equation for the shoreline position, x, took the form of the diffusion equation.

$$\frac{\partial x}{\partial t} = \frac{q_o}{D} \frac{\partial^2 x}{\partial y^2} \qquad (5)$$

where: $x$ = longshore coordinate

$q_o$ = derivative of the longshore transport rate with respect to the angle of wave incidence

$D$ = depth out to which sand transport is negligible

Equation 5 has been solved for a variety of boundary and initial conditions as detailed in Bakker (1968), Bakker et al. (1970), and LeMehaute and Soldate (1977).

Multiple line models, depicting two or more depth contours, provide information on the local beach slope (or profile) and provide a mechanism for incorporating on-offshore sediment movements. Bakker (1968) developed the first two-line model where the coupled equations governing the position of each line were as follows

$$(D_B+D_1)\frac{\partial x_1}{\partial t} = \frac{-\partial Q_1}{\partial y} - q_x \tag{6}$$

and

$$D_2\frac{\partial x_2}{\partial t} = \frac{-\partial Q_2}{\partial y} + q_x \tag{7}$$

where (referring to Figure 3):

$x_1$ = distance of the first (MSL) contour line from the base line

$x_2$ = distance of the second ($D_1$) contour line from the base line

$D_1$ = depth associated with the second contour line

$D_B$ = height of the sand berm above MSL

$D_1 + D_2$ = depth below which sand transport is negligible

$Q_1$ = longshore sediment transport rate associated with the $x_1$ contour line

$Q_2$ = longshore sediment transport rate associated with the $x_2$ contour line

$q_y$ = offshore sediment transport rate from the $x_1$ to the $x_2$ contour line

Solution of Bakker's two-line model is most easily accomplished numerically. The resulting solutions correctly simulate local beach bathymetry changes which occur around a groin, including the steepening of the profile on the upcoast side and the flattening of the profile on the downcoast side (see Figure 4).

Recently, Perlin and Dean (1983) have developed a six-line model which is a logical extension of Bakker's two-line model. Although the six-line model more accurately represents the shape of the beach profile than the two-line model, neither model accurately predicts the beach profile changes associated with changes in the incident wave field.

## ENERGETICS SURFZONE SEDIMENT TRANSPORT EQUATIONS

Because of the complexity of the flow field in the surfzone and the simplicity of Bagnold's energetics-based sediment transport concepts, the latter has found widespread application in modeling surfzone sediment transport (Inman and Bagnold, 1963; Komar, 1977; Thornton, 1973; Bowen, 1980; Bailard, 1981). The following is a brief summary of Bagnold's (1963, 1966) sediment transport model for streams, and Bailard's (1981, 1983) application of this model to sediment transport in the surfzone.

### Generalized Transport Equation

Bagnold's energetics-based sediment transport model for streams assumes that the transport occurs as two distinct modes, bedload and suspended load. Bedload sediment transport occurs as a thin granular-fluid layer which is supported by the bed via grain to grain collisions. Suspended load transport takes place above the bedload layer, where the sediment is supported by fluid turbulence. In both cases, energy is expended by the stream in transporting sediment, thus an analogy can be made of a stream acting as a sediment transport machine. Bagnold assumed that the rate of sediment transport was proportional to the rate of energy dissipation of the stream. The proportionality constants associated with the bedload and suspended load transports were termed efficiency factors, assumed to be constant. Bailard (1981) generalized Bagnold's energetics-based stream model for time-varying flow over an arbitrarily sloping bottom. The resulting sediment transport equation was

$$\langle \vec{i}_t \rangle = \rho\, C_f \frac{\varepsilon_B}{\tan\phi} \left[ \langle |\vec{u}_t|^2\, \vec{u}_t \rangle - \frac{\tan\beta}{\tan\phi} \langle |\vec{u}_t|^3 \rangle\, \hat{i} \right]$$

bed load

$$+ \rho\, C_f \frac{\varepsilon_s}{W} \left[ \langle |\vec{u}_t|^3\, \vec{u}_t \rangle - \frac{\varepsilon_s}{W} \tan\beta \langle |\vec{u}_t|^5 \rangle\, \hat{i} \right]$$

suspended load (8)

where: $\vec{i}_t$ = instantaneous sediment transport rate vector

$\rho$ = density of water

$C_f$ = drag coefficient of the bed

$\varepsilon_B$ = bedload efficiency factor

$\phi$ = internal angle of friction for the sediment

$\tan\beta$ = bed slope

$\varepsilon_s$ = suspended load efficiency factor

$W$ = fall velocity of the sediment

$\vec{u}_t$ = instantaneous nearbottom fluid velocity vector

$\hat{i}$ = unit vector directed upslope

$\langle\ \rangle$ = a time-average quantity

An important feature of the above equation is that the bedload transport (first bracketed quantity) and the suspended load transport (second bracketed quantity) consist of primary components directed parallel to the instantaneous fluid velocity vector, and secondary components directed downslope. The latter are associated with the downslope component of the sediment load.

## Local Surfzone Transport Equations

Figure 5 depicts a plane contour beach with the x-axis directed shoreward and normal to the beach, and the y axis directed parallel to the beach. The slope of the beach is $\tan\beta$. An incident wave with deep water angle, $\alpha_o$, is assumed to have a local wave angle $\alpha$.

For simplicity, the nearbottom velocity field is assumed to be composed of an oscillatory component $\tilde{u}$ oriented at an angle $\alpha$ to the x-axis, and steady velocity components u and v directed in the x and y directions, respectively. The total velocity vector $\vec{u}_t$ becomes

$$\vec{u}_t = (\tilde{u} \cos \alpha + u)\, \hat{i} + (\tilde{u} \sin \alpha + v)\, \hat{j} \tag{9}$$

In addition, the oscillatory velocity component is assumed to be composed of a primary component $u_m$ with frequency $\sigma$ and higher harmonics $u_{m2}$ with frequencies $2\sigma$, etc., so that

$$\tilde{u} = u_m \cos \sigma t + u_{m2} \cos 2\, \sigma t + \ldots \tag{10}$$

Substitution of Equation 9 into Equation 8, and assuming that

$u/u_m \ll 1$

$v/u_m \ll 1$

$\cos \alpha \cong 1$

$\langle i_x \rangle \ll \langle i_y \rangle$

then the idealized on-offshore and longshore transport equations become

$$\begin{aligned}\langle i_x \rangle = {} & \rho\, C_f\, u_m^3 \frac{\varepsilon_B}{\tan \phi} \left[ \Psi_1 + \frac{3}{2} \delta_u - \frac{\tan \beta}{\tan \phi} u_3^* \right] \\ & + \rho\, C_f\, u_m^4 \frac{\varepsilon_s}{W} \left[ \Psi_2 + 4\, u_3^*\, \delta_u - \varepsilon_s \frac{u_m}{W} \tan \beta\, u_5^* \right]\end{aligned} \tag{11}$$

and

$$\begin{aligned}\langle i_y \rangle = {} & \rho\, C_f\, u_m^3 \frac{\varepsilon_B}{\tan \phi} \left[ \frac{\delta_v}{2} + \delta_v{}^3 + \frac{\tan \beta}{\tan \phi} u_3^* \tan \alpha \right] \\ & + \rho\, C_f\, u_m^4 \frac{\varepsilon_s}{W} \left[ \delta_v\, u_3^* + \frac{u_m}{W} \varepsilon_s \tan \beta\, u_5^* \tan \alpha \right]\end{aligned} \tag{12}$$

where

$$\delta_v = v/u_m$$

$$\delta_u = u/u_m$$

$$\Psi_1 = \langle \tilde{u}^3 \rangle \Big/ u_m^3$$

$$\langle \left| \vec{u}_t \right|^3 \vec{u}_t \rangle \cong \left( \Psi_2 + 3\ \delta_u\ u_3^* \right) u_m^4$$

$$\Psi_2 = \langle \left| \tilde{u} \right|^3 \tilde{u} \rangle \Big/ u_m^4$$

$$u_3^* = \langle \left| \vec{u}_t \right|^3 \rangle \Big/ u_m^3$$

$$u_5^* = \langle \left| \vec{u}_t \right|^5 \rangle \Big/ u_m^5$$

Equations 11 and 12 describe the local on-offshore and longshore transport rates as a function of local wave, current, beach, and sediment parameters. Combined with nearbottom current meter records, these equations can be used to predict the local sediment transport rate vector as a function of time. For modeling purposes, however, the distribution and spatially integrated values of the time-averaged sediment transport rate are of greater interest. Predicting these transport rates as a function of the incident wave field requires the use of additional surfzone model theory.

Considering the longshore transport Equation 12, all of the free parameters can be predicted from linear wave shoaling/refraction theory and one of several radiation stress-based longshore current models which are available (e.g., Longuett-Higgins, 1970). In its full complexity, this procedure results in a complex expression for the longshore sediment transport rate which must be numerically evaluated (Bailard, 1981). As a result, the model is difficult to apply and the results are not easily generalized. In the next section, the approach used by Bailard (1981) will be simplified yielding a less complex and more easily applied longshore transport model.

Equation 11, which predicts the on-offshore transport, is more difficult to apply than the longshore transport Equation 12 because the wave velocity moments, $\Psi_1$ and $\Psi_2$, and the mean velocity, $\delta_u$, cannot be predicted from existing surfzone model theory. The approach taken in the present study was to estimate these parameters from measured field data and correlate their values with the incident wave characteristics. Details of this procedure will be discussed in a later section.

## SPATIALLY INTEGRATED LONGSHORE TRANSPORT

The ubiquity of the wave power equation (Equation 1) suggested that the present longshore transport model should take the form of a modification to the wave power equation. Following the approach by Bailard (1981), an expression was sought for the wave power coefficient, K, as a function of incident wave, beach, and sediment parameters.

The principal obstacles to obtaining a close-form solution for K were the complex expressions for the wave velocity moments, $u_3^*$ and $u_5^*$. Guza and Thornton (1981) have shown, however, that for weak mean currents and a Gaussian orbital velocity distribution, $u_3^*$ and $u_5^*$ are constants equal to 0.58 and 1.13, respectively. Field measurements of $u_3^*$ and $u_5^*$ (Bailard, 1983) support this finding with measured values equal to 0.60 to 1.25, respectively. Consequently, an important simplifying assumption made in the present study was that $u_3^*$ and $u_5^*$ are constants equal to the above measured values.

The longshore current model used in the present development was the radiation stress-based model by Ostendorf and Madsen (1979). In order to obtain a tractable solution, a number of simplifications were made with regard to the model's provisions for finite breaker heights and finite longshore current strengths. The resulting expression for the longshore current, v, was as follows

$$v = v_c \, v^* \tag{13}$$

where $v_c$ is a characteristic longshore current strength and $v^*$ is the dimensionless longshore current distribution expressed as follows ($P \neq 0.4$)

$$v^* = C_1 \left( \frac{1 - C_2}{C_2 - C_3} \right) x^{*^{C_3}} + C_1 x^* \qquad x^* \leq 1$$

$$v^* = C_1 \left( \frac{1 - C_3}{C_2 - C_3} \right) x^{*^{C_2}} \qquad x^* > 1 \tag{14}$$

The parameters $C_1$, $C_2$, and $C_3$ are functions of the lateral mixing parameter, P, as follows

$$C_1 = (1 - 2.5\ P)^{-1}$$

$$C_2 = -\frac{1}{8} - \left( \frac{1}{64} + \frac{1}{P} \right)^{1/2}$$

$$C_3 = -\frac{3}{4} + \left( \frac{9}{16} + \frac{1}{P} \right)^{1/2} \tag{15}$$

and the dimensionless surfzone position, $x^*$, is defined as

$$x^* = \frac{x - x_s}{x_b - x_s} \tag{16}$$

where $x_s$ and $x_b$ are the positions of the shoreline and the break-point respectively.

The characteristic longshore current strength, $v_c$, is defined as

$$v_c = \delta_c\ u_{mb} \tag{17}$$

where $u_{mb}$ is the oscillatory velocity magnitude at the breakpoint obtained from shallow water wave theory, i.e.,

$$u_m = \frac{\gamma}{2} \sqrt{g\ h} \tag{18}$$

and $\delta_c$ is the relative longshore current strength parameter defined as

$$\delta_c = \frac{5\pi \tan \Delta \sin \alpha_b}{8 C_f} \tag{19}$$

Note that in Equations 18 and 19, $\tan \Delta$ is equal to the average beach slope, $\tan \beta$, modified for wave setup, i.e.,

$$\tan \Delta = \frac{\tan \beta}{1 + \frac{3}{8} \gamma_b^{\,2}} \tag{20}$$

and $\gamma_b$ is the ratio of the surfzone wave height, H, to water depth, h, defined as

$$\gamma = \gamma_b \qquad x^* \leq 1$$

$$\gamma = \gamma_b \left(\frac{h_b}{h}\right)^{5/4} \qquad x^* > 1 \tag{21}$$

and $\gamma_b$ is assumed to be equal to 0.8.

Combining the longshore current Equation 13 and the local longshore transport Equation 12, and assuming that Snells Law applies, the following equation is obtained for the cross-shore distribution of the longshore transport rate

$$\begin{aligned} \langle i_y \rangle = {} & \rho\, C_f\, u_{mb}^3 \frac{\varepsilon_\beta}{\tan \phi} \left[ \frac{1}{2} \left(\frac{\gamma}{\gamma_b}\right)^2 \delta_c\, v^*\, x^* + \delta_c^{\,3}\, v^{*3} \right. \\ & \left. + \frac{\tan \beta}{\tan \phi} \left(\frac{\gamma}{\gamma_b}\right)^3 u_3^*\, x^{*2} \tan \alpha_b \right] \\ & + \rho\, C_f\, u_{mb}^4 \frac{\varepsilon_s}{W} \left[ \left(\frac{\gamma}{\gamma_t}\right)^3 \delta_c\, u_3^*\, v^*\, x^{*3/2} \right. \\ & \left. + \tan \beta \frac{u_{mb}}{W} \varepsilon_s \left(\frac{\gamma}{\gamma_b}\right)^5 u_5^*\, x^{*3} \tan \alpha_b \right] \end{aligned} \tag{22}$$

The spatially integrated longshore transport rate, $I_\ell$, may be obtained by integrating Equation 19,

$$I_\ell = \int_{-\infty}^{x_s} \langle i_y \rangle \, dx = \frac{h_b}{\tan \Delta} \int_{o}^{\infty} \langle i_y \rangle \, dx^* \tag{23}$$

Noting that $K = I_\ell / P_\ell$, $\tan \alpha_b \cong \sin \alpha_b$, and

$$\rho \, C_f \frac{h_b}{\tan\Delta} \, u_{mb}^3 \, \delta_c = \frac{5\pi}{8} \gamma_b \, P_\ell \tag{24}$$

then the wave power coefficient K can be expressed as

$$K = \varepsilon_B \, K_1 + \varepsilon_S \, K_2 + \varepsilon_S^2 \, K_3 \tag{25}$$

where:

$$K_1 = \frac{5\pi}{8} \frac{\gamma_b}{\tan \phi} \left\{ \frac{C_1}{2\,(C_2 - C_3)} \left[ \frac{1 - C_2}{C_3 + 2} + \frac{C_2 - C_3}{3} - \frac{1 - C_3}{C_2 - 1/2} \right] \right.$$

$$+ \frac{25\,\pi^2}{256} \left(1 + \frac{3}{8}\gamma_b^2\right)^{-2} \left(\frac{\tan \beta}{C_f}\right)^2 \sin^2 2\alpha_b \, C_1^3 \left[ \frac{\left(\frac{1 - C_2}{C_2 - C_3}\right)^3}{3\,C_3 + 1} \right.$$

$$\left. + \frac{3\left(\frac{1 - C_2}{C_2 - C_3}\right)^2}{2\,C_3 + 2} + \frac{3\left(\frac{1 - C_2}{C_2 - C_3}\right)}{C_3 + 3} + \frac{1}{4} - \frac{\left(\frac{1 - C_3}{C_2 - C_3}\right)}{3\,C_2 + 1} \right]$$

$$\left. + \frac{8}{15\,\pi} \frac{\tan \beta}{\tan \phi} \left(\frac{C_f}{\tan \beta}\right)\left(1 + \frac{3}{8}\gamma_b^2\right) u_3^* \right\} \tag{26}$$

$$K_2 = \frac{5\pi}{8} \gamma_b u_3^* \frac{u_{mb}}{W} \left(\frac{C_1}{C_2 - C_3}\right) \left[\frac{1 - C_2}{C_3 + 5/2} + \frac{C_2 - C_3}{7/2} - \frac{1 - C_3}{C_2 - 5/4}\right] \quad \text{. . . . . . . . (27)}$$

and

$$K_3 = \frac{25}{36} \gamma_b \tan\beta \left(\frac{c_f}{\tan\beta}\right) \left(1 + \frac{3}{8}\gamma_b^2\right) u_5^* \left(\frac{u_{mb}}{W}\right)^2 \quad (28)$$

Further simplifications to Equations 26 through 28 are possible using the results of Komar (1976). Komar analyzed the results of laboratory and field measurements of longshore currents and found that P was approximately equal to 0.2 and the ratio $\tan\beta/c_f$ was approximately equal to 7. Introducing these assumptions into Equations 26 through 28 the following results are obtained

$$K_1 = 0.385 + 20 \sin^2 2\alpha_b + 0.074 \tan\beta \quad (29)$$

$$K_2 = 0.228 \frac{u_{mb}}{W} \quad (30)$$

$$K_3 = 0.123 \tan\beta \left(\frac{u_{mb}}{W}\right)^2 \quad (31)$$

The only free parameters remaining in Equation 25 that must be specified are the bedload and suspended load efficiency factors. Bagnold (1966) found that $\varepsilon_B$ and $\varepsilon_S$ were equal to 0.15 and 0.01 respectively for stream flow. In the present case, these factors were estimated from laboratory and field measurements of the longshore transport rate.

The laboratory data used to estimate $\varepsilon_B$ and $\varepsilon_S$ were selected data from Saville (1949, 1950) and from Shay and Johnson (1951). The field data were from Bruno et al. (1980), Dean et al. (1983), Komar and Inman

(1970), Kraus et al. (1983), and Moore and Cole (1960). Average characteristics of these data sets are summarized in Table 1.

Initially, estimates for $\varepsilon_B$ and $\varepsilon_S$ were sought using a nonlinear least squares estimation procedure. It was quickly found that the contribution from $K_3$ was negligible. Eliminating $K_3$ from Equation 22 yields an equation linear in $\varepsilon_B$ and $\varepsilon_S$, thus a simpler multiple linear regression analysis was used. The resulting estimated values for $\varepsilon_B$ and $\varepsilon_S$, along with their 95% confidence intervals, were

$$\varepsilon_B = 0.13 \pm 0.009$$

$$\varepsilon_S = 0.032 \pm 0.004 \tag{32}$$

Utilizing these values in Equation 25 the final equation for the wave power coefficient, K, becomes

$$K = \underbrace{0.050 + 2.6 \sin^2 2\alpha_b + .0096 \tan \beta}_{\text{Bedload}} + \underbrace{0.0073 \frac{u_{mb}}{W}}_{\text{Suspended load}} \tag{33}$$

Figure 6 shows a plot of the observed values of K versus the estimated values of K. With the exception of the El Moreno Beach data from Komar and Inman (1970) and the single Moore and Cole (1960) data point, the fit is relatively good.

<u>Spatially Averaged On-offshore Transport Equation</u>

Equation 11 predicts the local on-offshore sediment transport rate as a function of the local wave, current, beach, and sediment parameters. The steady onshore current is characterized by $\delta_u$, while the waves are characterized by the orbital velocity magnitude, $u_m$, the total velocity moments, $u_3^*$ and $u_5^*$, and the two skewness parameters, $\Psi_1$ and $\Psi_2$. The latter describe the asymmetry of the orbital velocity field as characterized by the brief but intense onshore flow occurring beneath the wave crest and the longer but less intense return flow occurring beneath the wave trough.

In principal, $u_3^*$, $u_5^*$, $\delta_u$, and $u_m$ can be predicted from existing surfzone wave theory, while $\Psi_1$ and $\Psi_2$ cannot. For example, $u_3^*$ and $u_5^*$ have been shown to be approximately constant, while $U_m$ may be assumed to be a function of the local water depth (Equation 18). Moreover, several models have recently been proposed to describe $\delta_u$ as a function of the incident wave parameters (e.g. Svendsen, 1984). In practice, application of these models to field conditions is difficult. For example, field measurements (Guza and Thornton, 1981) show $u_m$ to be nearly constant across the surfzone instead of decaying as $h^{1/2}$ as suggested by the breaking wave theory. In addition, the models for $\delta_u$ are too complex for easy application within the context of developing a simple on-offshore transport model. As a result, the approach taken in the present study was to empirically relate measured values of $u_m$, $\delta_u$, $\Psi_1$, and $\Psi_2$ with incident wave properties.

The field data sets used to estimate the above parameters were obtained from the Nearshore Sediment Transport Study (NSTS) experiments at Torrey Pines Beach, Calif., and Leadbetter Beach, Calif. (Gable 1979, 1980). Each experiment consisted in part of the simultaneous measurements of incident wave characteristics, near bottom surfzone velocities, and beach profile changes. The surfzone velocities were measured using electromagnetic current meters arranged in a cross-shaped pattern. One line of current meters was aligned parallel to the beach, while another line of current meters was aligned perpendicular to the beach. In the present study, only data from the shore-perpendicular current meters were used.

The data sets from the two beaches differed significantly in terms of the beach slope (0.02 for Torrey Pines Beach and 0.038 for Leadbetter Beach) and the incident wave characteristics. Figure 7 shows that at Torrey Pines Beach, the significant wave height varied between 0.8 and 2.0 meters, while the significant wave period varied between 9 and 12 seconds. At Leadbetter Beach, the significant wave height varied between 0.2 and 0.8 meter, while the significant wave period varied between 5 and 20 seconds.

Initially, it was hoped that a set of empirical equations could be developed to predict $u_m$, $\delta_u$, $\Psi_1$, and $\Psi_2$ as a function of surfzone position, incident wave properties, and the average beach slope. In this way, the distribution of the on-offshore sediment transport could be predicted as a function of the incident wave characteristics for a particular beach slope.

In order to determine whether or not a similarity type distribution existed, data was plotted as a function of $h/L_o$, $x^*$, and the Ursell number, $H/k_o^2h^3$, where H is the wave height, h is the water depth, $L_o$ is the deep water wave length, and $k_o$ is the deep water wave number. None of the plots showed a significantly reduced degree of scatter in the data relative to plots of these parameters versus the depth, h.

Because of the poor results of the above procedure, a less ambitious approach was taken whereby the spatially averaged values of $u_m$, $\delta_u$, $\Psi_1$, and $\Psi_2$ were correlated with gross properties of the incident waves. The latter included the incident significant wave height, incident significant wave steepness, and the surfzone similarity parameter defined as

$$\varepsilon_o = \frac{\tan \beta}{\left(H_o/L_o\right)^{1/2}} \tag{34}$$

Although the results were not conclusive, the best correlation was obtained when the variables were plotted against the incident significant wave height.

Figures 8 through 11 show plots of $u_m$, $\delta_u$, $\Psi_1$, and $\Psi_2$ versus significant wave height for each data set. These plots suggested that each of the surfzone parameters could be fitted to a regression equation of the following form

$$\text{parameter} = (a \tan \beta + b)\, H_s + (c \tan \beta + d) \tag{35}$$

where: $H_s$ = signficant wave height, meters

$\tan \beta$ = average beach slope within the surfzone

Expanding Equation 35 and using a multiple linear regression analysis, estimates for the coefficients a through d were obtained for each variable. The results are listed in Table 2, while the linear regression curves are shown in Figures 7 through 10.

The regression equations for the parameters $u_m$, $\delta_u$, $\Psi_1$, and $\Psi_2$ can be combined with the on-offshore sediment transport equation (Equation 11) to predict the spatially-averaged on-offshore sediment transport rate in the surfzone as a function of the incident wave height and the average beach slope.

Figure 12 shows a plot of the onshore transport rate as a function of the incident significant wave height for three different beach slopes. Free parameter inputs used to generate this figure were

$$C_f = 0.005$$

$$W = 0.04 \text{ m/sec}$$

$$S_s = 2.65$$

$$N_o = 0.63$$

$$\varepsilon_B = 0.13$$

$$\varepsilon_S = 0.032$$

The behavior of the on-offshore sediment transport model as depicted in Figure 11 shows several realistic features. First, gradual onshore sand movement occurs when the waves are small, while rapid offshore movement occurs with larger waves. This feature helps to explain why beaches can erode rapidly under the attack of large winter waves. In contrast, beaches accrete slowly under the action of small summer waves.

A second feature of the model is that an equilibrium wave height exists which is a function of the average beach slope. With steep beaches, the equilibrium wave height is small, while with flat beaches the equilibrium wave height is larger. A corollary to this finding is that given sufficient time, a beach will seek a beach slope in equilibrium with the incident wave height. Figure 13 shows a plot of the equilibrium beach slope as a function of the incident significant wave height. Small waves produce a steep beach, while large waves produce a flat beach. This is in keeping with field observations.

One feature which is lacking in the model is any significant variation in the equilibrium beach slope with sediment fall velocity (i.e., sediment size). Varying the sediment fall velocity by an order of magnitude has the effect of changing the magnitude of the on-offshore transport rate (i.e., the rate at which the beach responds to a change in wave height), but not the equilibrium wave height. Long-term field observations (e.g., U.S. Army CERC, 1977) have shown a link between sediment size and beach slope, however, this may be due to threshold of motion or percolation effects, neither of which are included in the present model.

It is of interest to compare the Bakker/Swart on-offshore sediment transport equation (Equation 4) with the present model. One way to accomplish this is to assume normally incident waves and utilize a degenerative form of the two-line model equations. Assuming $Q_{y1}$ and $Q_{y2}$ are zero, Equations 5 and 6 become

$$(D_B+D_1)\,\frac{\partial\, x_1}{\partial t} = -\, q_x \qquad (36)$$

$$D_2\,\frac{\partial\, x_2}{\partial t} = q_x \qquad (37)$$

Solutions to the above equations are expressed in terms of the position of the contour lines $x_1$ and $x_2$, as a function of time. For present purposes, it is more convenient to express the solution in terms of the time history of the average beach slope expressed as

$$\tan\beta = \frac{2\,D_1}{x_2 - x_1} \qquad (38)$$

Utilizing the present model, Equations 36 and 37 can be combined with the on-offshore transport model described by Equations 11 and 35. Assuming values for the initial beach slope, the sediment fall velocity, the beach depths $D_1$, $D_2$, $D_b$, and the drag coefficient for the bed, $C_f$, these equations can be time-stepped ahead to predict the evolution of the beach slope subject to specific incident wave conditions.

Figures 14 and 15 show the results of this procedure utilizing three different initial beach slopes and two different incident wave heights. Input conditions for these figures were as follows

| Variable | Figure 14 | Figure 15 |
|---|---|---|
| $\tan \beta_i$ | 0.03; 0.02; 0.015 | 0.03; 0.02; 0.010 |
| $H_s$ | 0.9 meter | 0.5 meter |
| $C_f$ | 0.005 | 0.005 |
| W | 0.04 m/sec | 0.04 m/sec |
| $D_1$ | 1.5 meters | 0.5 meter |
| $D_B$ | 1.0 meter | 1.0 meter |
| $D_2$ | See Equation (45) | |

Note that the equilibrium beach slopes corresponding to the wave heights 0.9 and 0.5 meter are 0.02 and 0.03 respectively. Generalizing these results, it is seen that the flattening of an oversteep beach occurs more quickly than the steepening of an overflattened beach. The rate of change of the beach slope at a particular moment depends on both the initial beach slope and the incident wave height.

Solution of Equations 36 and 38 using the Bakker/Swart on-offshore transport model (Equation 4) produces an analytic expression for the evolution of the beach slope as a function of time (see Appendix A). The resulting expression for the beach slope is

$$\tan \beta = \frac{\tan \beta_i \tan \beta_e}{\tan \beta_i \left(1-e^{-Bt}\right) + \tan \beta_e e^{-Bt}} \qquad (39)$$

where:

$$B = \frac{Coff}{D_B + D_1} + \frac{Coff}{D_2} \tag{40}$$

$\tan \beta_i$ = initial beach slope

$\tan \beta_e$ = equilibrium beach slope

Using the same input conditions discussed above, except that the equilibrium beach slope is specified as opposed to the incident wave height and Coff = 0.263 m/day, the resulting time history of the beach slope is plotted in Figures 14 and 15.

Comparing these results with the results presented in Figures 13 and 14, several differences are evident. First, in the Bakker/Swart model, the beach slope response curve is symmetrical with regards to erosion or accretion. The rate at which the beach slope changes is a direct function of the parameter Coff, and the depths $D_1$, $D_2$, and $D_b$. Bakker (1970) suggested that Coff should be between 0.00274 and 0.0274 m/day, while Perlin and Dean (1983) suggested a value closer to 0.263 m/day.

The above results suggest that the present model conforms more closely to field observations of beach profile response than does Bakker's (1970) model. The latter could be made more realistic by allowing Coff to be a function of the incident wave height, the fall velocity of the sediment, and the initial beach slope. This procedure was in fact used by Swart (1974), except that his model describes the evolution of the complete beach profile.

## LINE MODEL APPLICATION

The present longshore and on-offshore transport models were incorporated into a simple two-line model to predict the evolution of nearshore bathymetry around a single groin. Figure 3 shows the important elements of the line model. The shoreline (or mean sea level) contour is described by $x_1$ line, while the $D_1$ depth contour is described by the

$x_2$ line. The depth, $D_1 + D_2$ is assumed to be equal to the depth at which active sand transport ceases. The elevation $D_B$ describes the height of the beach berm line above mean sea level.

The basic relationships governing the line model include the sediment continuity equation, the longshore transport rate, and the on-offshore transport rate. Additional details discussed below include breaking wave assumptions, the depth of active sand transport, boundary condition, and the development of the controlling finite difference equations.

## Basic Equations

The sand continuity equations (Equations 5 and 6) describe the change in shore line position as a function of the on-offshore transport rate and the divergence of the longshore transport rate. In these equations, the on-offshore transport represents an exchange of sand across the $D_1$ contour line.

The on-offshore transport model developed in the previous section describes the spatially-averaged on-offshore sand transport in the surfzone. No details regarding the distribution of this transport are predicted, nor is a precise definition given for the region of application in the surfzone. Thus, for simplicity it was assumed that the predicted on-offshore transport rate was equal to that crossing the $D_1$ depth contour (i.e., just that required by the two-line model).

In a two-line model, the longshore transport rate is divided into two parts, $Q_1$ and $Q_2$. $Q_1$ includes all of the longshore transport occurring shoreward of the $D_1$ depth contour, while $Q_2$ includes all the transport occurring seaward of this contour. As a result, this division of the longshore transport rate can be expressed as follows

$$Q_1 = \eta_1 Q_\ell \quad \text{and} \quad Q_2 = (1-\eta_1) Q_\ell \tag{41}$$

where: $Q_\ell$ = total longshore transport rate given by Equations 1, 3, and 33

The parameter $\eta_1$ is defined as

$$\eta_1 = \frac{\int_0^{x_1^*} \langle i_y \rangle dx^*}{\int_0^{\infty} \langle i_y \rangle dx^*} \qquad (42)$$

where: $x_1^* = D1/h_b$

An analytical expression for $\eta_1$ is derived in Appendix B.

Breaking Wave Assumptions

Wave input conditions for the line model consist of the incident significant wave height, the significant wave period, and the deep water incident wave angle (measured counterclockwise from the y axis). As the waves progress into shallow water, refraction and shoaling processes cause the wave angle and the wave height to change. Wave diffraction is also important in the vicinity of the groin. LeMehaute and Soldate (1980) and Kraus and Harikai (1983) have developed methods for incorporating wave diffraction effects into single-line model methodology. In the present model development, however, wave diffraction effects were judged to be a complication which could be added later.

As the waves propagate into shallower and shallower water, the ratio of wave height to water depth eventually becomes so large that the waves break. Following the development of LeMehaute and Soldate (1980), the breaker height and breaker angle are computed from the deep water wave height, $H_o$, and the deep water wave angle (relative to the x axis), $\alpha$, in the following manner

$$H_b = K_r K_s H_o \qquad (43)$$

and

$$\alpha_{b_i} = \alpha_{o_i} (0.25 + 5.5 H_o/L_o) \quad (i=1,2) \qquad (44)$$

where:

$$\alpha_{o_i} = \alpha - \tan^{-1}\left(\frac{\partial x_i}{\partial y}\right) \qquad (i=1,2)$$

$$K_r = \left(\frac{\cos \alpha_{o1}}{\cos \alpha_b}\right)^{1/2}$$

$$K_s = 0.3 \left(L_o/H_o\right)^{1/3}$$

$$H_o = \frac{H_s}{1.418}$$

Note also that in the above equations, the subscripts 1 and 2 refer to the $x_1$ and $x_2$ contour lines, respectively.

Depth of Closure

Following the approach of Kraus and Harikai (1983), the depth of closure model developed by Hallermeier (1978) was used to describe the seaward boundary of the zone where significant longshore and intense on-offshore transport takes place. Hallermeier's model expresses the critical depth, $D_c = D_1 + D_2$ as

$$D_c = 2.28\ H_s - 68.5\ (H_s^{\ 2}/gT_s^2) \qquad (45)$$

where $D_c$ and $H_s$ are expressed in meters, g in m/sec$^2$, and $T_s$ in seconds.

Boundary and Initial Conditions

The initial conditions for the model are completely arbitrary, however for simplicity, the shoreline is assumed to be initially parallel to the y axis, with

$$y_1 = 0$$

and

$$y_2 = 2\, D_1/\tan\, \beta_i \tag{46}$$

where: $\tan\, \beta_i$ = the initial beach slope

The boundary conditions for the model are specified at both the groin and the far upcoast and downcoast positions. At the latter positions, it is assumed that

$$\frac{\partial\, x_i}{\partial y} = 0 \quad i=1,2 \tag{47}$$

At the groin, the boundary condition is specified in terms of the rate of longshore transport. It is assumed that sand is allowed to bypass the groin in proportion to $\eta_g$, which is defined as follows. Letting the dimensionless groin length, $x_g^* = h_g/h_b$, where $h_g$ is the depth of water at the end of the groin, then the coefficient $\eta_g$ can be evaluated using Equation 42. Having specified $\eta_g$, the boundary condition at the groin is specified as

If $\eta_g <= \eta_1$ then:

$$\eta_1' = \eta_1 - \eta_g \text{ and } \eta_2' = \eta_2$$

If $\eta_1 < \eta_g <= \eta_2$ then: (48)

$$\eta_i = 0 \quad \text{and } \eta_2' = \eta_2 - \eta_g$$

If $\eta_g > \eta_2$ then:

$$\eta_1' = 0 \quad \text{and} \quad \eta_2' = 0$$

where: the superscript ' = value of $\eta_1$ or $\eta_2$ at the groin

Finite Difference Solution

Following the approach of Perlin and Dean (1978), an explicit solution utilizing a time-marching space and time-staggered step procedure was used. This procedure involves holding the shoreline orientation fixed for one-time step (from nt to (n+1)t) and calculating the longshore and on-offshore sand transports. The sand transport is then held fixed over a time step (from (n+1/2)t to (n+3/2)t) and the shoreline changes are determined. The finite difference equations are

$$Q_{1_i}^{n+1} = \eta_1^{n+1} K_i^{n+1} \frac{\left(H_b^2\right)_i^n}{8} \frac{Cn_{b_i}^n}{\left(S_s - 1\right) N_o} \sin \alpha_{b_{1_i}}^{n+1/2} \cos \alpha_{b_{1_i}}^{n+1/2} \qquad (49)$$

$$Q_{2_i}^{n+1} = \eta_2^{n+1} K_i^{n+1} \frac{\left(H_b^2\right)_i^n}{8} \frac{Cn_{b_i}^n}{\left(S_s - 1\right) N_o} \sin \alpha_{b_{2_i}}^{n+1/2} \cos \alpha_{b_{2_i}}^{n+1/2} \qquad (50)$$

$$q_{x_i} = \frac{\langle i_x \left(K_R H_s^n, \tan \beta^{n+1/2}\right)_i \rangle}{\rho\, g \left(S_s - 1\right) N_o} \qquad (51)$$

$$x_{1_i}^{n+3/2} = x_{1_i}^{n+1/2} - \frac{\Delta t}{\left(D_B + D_1\right) \Delta y} \left[Q_{1_{i+1}}^{n+1} - Q_{1_i}^{n+1} + \Delta y\, q_{x_i}^{n+1}\right] \qquad (52)$$

$$x_{2_i}^{n+3/2} = x_{2_i}^{n+1/2} - \frac{\Delta t}{D_2 \, \Delta y} \left[ Q_{2_{i+1}}^{n+1} - Q_{2_i}^{n+1} - \Delta y \, q_{x_i}^{n+1} \right] \tag{53}$$

$$\alpha_{b_{j_i}}^{n+1/2} = \alpha_{o_{j_i}}^{n+1/2} \left( 0.25 + 5.5 \, H_o/L_o \right) \qquad j=1,2 \tag{54}$$

$$\alpha_{o_{j_i}}^{n+1/2} = \alpha - \tan^{-1} \frac{\left( x_{j_i}^{n+1/2} + x_{j_{i-1}}^{n+1/2} \right)}{\Delta y} \qquad j=1,2 \tag{55}$$

where the superscripts denote the time level at which the variable is evaluated and the subscripts denote the space step at which the variable is evaluated.

## Example Solutions

Two examples are presented which illustrate the shoreline evolution around a newly constructed groin superimposed on an overall changing beach profile. In the first example, the initial beach slope is steeper than the equilibrium beach slope. In the second example, the initial beach slope is flatter than the equilibrium beach slope. Input conditions for the two examples are as follows:

Example 1:

$H_s$ = 2.12 meters; $T$ = 8 seconds; $\alpha$ = -25 degrees

$\tan \beta_i$ = 0.02; $D_b$ = 1.0 meters; $D_1$ = 1.5 meters

$W$ = 0.04 m/sec; $\Delta y$ = 100 meters; $\Delta t$ = 3 hours

$x_{groin}$ = 100 meters

Example 2:

$H_s$ = 0.71 meter; $T$ = 8 seconds; $\alpha$ = -25 degrees

$\tan \beta_i = 0.15$; $D_b = 1.0$ meters; $D_1 = 0.75$ meters

$W = 0.04$ m/sec; $\Delta x = 100$ meters; $\Delta t = 12$ hours

$x_{groin} = 50$ meters

Figures 16 and 17 show the time evolution of the nearshore contours for Examples 1 and 2 respectively. Contour positions are plotted for times of 0, 1, 4, 16, and 64 days. Although the solution is shown for both sides of the groin, the contour line positions on the immediate down-draft side of the groin are not realistic because wave diffraction has been neglected.

In the immediate vicinity of the groin, Figures 16 and 17 show similar patterns of shoreline behavior, with a sand fillet forming on the upcoast side of the groin, and erosion occurring on the downcoast side. In addition, the offshore contours show a steepening of the beach on the upcoast side of the groin, and a flattening of the beach on the downcoast side. This behavior is in accordance with field observations.

Superimposed on the above general trends is an overall flattening of the entire beach in Figure 16 caused by a general offshore sand transport. In Figure 17, the beach is slowly steepening with a general onshore sand transport. The effect of these global changes is to exaggerate the effects of the local accretion and erosion around the groin in Example 1 and diminish the effects in Example 2.

## DISCUSSION

The primary advantages of the longshore and on-offshore transport models developed in this study are that they share a common energetics development and they are simple to apply. The equation for the wave power coefficient, K, involves parameters which are readily available from the usual longshore transport calculations. Separate accounting of the bedload and suspended load transports provides for a greater degree of flexibility in the wave power equation, allowing the equation to be used over a wider range of wave heights and sediment sizes. The on-offshore transport model is extremely easy to use, requiring only the incident wave height, the sediment fall velocity, and an average drag coefficient for the bed. When combined with a simple two-line shoreline evolution model, the above models can be used to predict beach profile and/or shoreline evolution around a groin.

The above models have several general and specific limitations. The general limitations are primarily related to gross simplifications made in deriving the energetics transport equations. First, the longshore and on-offshore sediment transport models are vertically integrated, thus ignoring the vertical structure of the sediment concentration and velocity fields. Recent field measurements (Jaffe, et al., 1984) suggest that the vertical distributions of these parameters are important.

Another general limitation of the energetics models is that the rate of energy dissipation (to which the transport rate is proportional) is explicitly related to the bottom boundary layer (i.e., the drag on the bottom). Contributions to the local rate of energy dissipation from the breaking wave at the surface are neglected, in spite of the fact that the latter has been shown to be dominant (Thornton and Guza, 1983). An implicit assumption used in the present model development was that the bottom boundary layer controls the upward flux (entrainment) of sediment. This assumption, however, is only conjecture. Possible improvements in the model might be to incorporate a variable drag coefficient to account for some of the effects of breaking waves; alternatively, the rate of energy dissipation could be directly associated with the local gradient in the energy flux (e.g., Thornton, 1973).

Ignoring sediment threshold of motion effects is another general limitation of the models. In the case of longshore transport of sand-sized sediments, under most field conditions threshold effects are negligible. However, threshold effects are probably significant in on-offshore transport under most field conditions. Similarly, they are probably important for longshore transport under laboratory conditions. The statistical method developed by Seymour (unpublished manuscript, 1984) may be one method of dealing with these effects.

Specific model shortcomings can be summarized as follows. The longshore transport distribution was derived using common spilling wave assumptions (e.g., the orbital velocity magnitude decreases as $h^{1/2}$). Field measurements on dissipative beaches, however, have shown that the total velocity variance, and thus the orbital velocity magnitude, is nearly constant across the surfzone. The longshore current model used in the present study is also largely unverified. Beach profile shapes have been shown to have a signficant effect on the distribution of the longshore current (Hudspeth and McDougal, 1984). Moreover, the proper formulation for the lateral mixing of momentum is still an unanswered question. As a result, the distribution of the longshore transport rate, as specified in the present model, should be viewed as approximate. The spatially integrated equation for the wave power coefficient, K, is probably more accurate, because uncertainties in the transport distribution have been hidden in the spatial integration.

The present on-offshore transport model is only a first step in describing the on-offshore transport. Although it is dynamically based, the empirically-derived regression equations for the wave velocity moments are restricted to a narrow range of conditions. In fact, the present model is unable to predict onshore sand motion for beach slopes in excess of approximately 0.035. Eventually, when theoretical or numerical models are developed to predict the wave velocity moments as a function of the incident wave field and the local water depth, it will be possible to predict the time-evolution of the entire beach profile. This would allow a direct comparison of the present dynamically based on-offshore transport model with Swart's (1974) empirically based beach profile model.

The line model methodology used in the present study is relatively well established. The contributions of this study relate to the incorporation of improved longshore and on-offshore sediment transport models. A further improvement of the present line model would be to include wave diffraction effects, using the techniques developed by LeMehaute and Soldate (1980) or Kraus and Harikai (1983). Extension of the model to three or more lines is not feasible at this time due to the limitations in the on-offshore transport model.

## CONCLUSIONS AND RECOMMENDATIONS

The following conclusions were reached as a result of this study.

1. An energetics-based longshore sediment transport model was developed which resulted in a modification to the wave power coefficient. Instead of being constant, K was found to be a function of the breaker angle, the beach slope, and the ratio of the orbital velocity magnitude at the breakpoint divided by the fall velocity of the sediment. These modifications were found to extend the range of applicability of the wave power equation to both laboratory and field conditions. Although the model describes the distribution of the longshore transport rate across the surfzone, this distribution should be applied with caution. Recent measurements of surfzone wave heights and currents suggest that a number of common assumptions used in the development of the present model, and most longshore current models, are not accurate.

2. An energetics-based on-offshore sediment transport model was developed which predicts the spatially averaged surfzone on-offshore transport rate as a function of the incident wave height, the average beach slope, and the sediment fall velocity. While the model remains unverified, it exhibits a qualitative behavior which corresponds to field observations of beach profiles. Additional work is needed to verify the existing model and to extend the model to predict the cross-shore distribution of the on-offshore sediment transport rate.

3. The above two sediment transport models were used to develop a two-line model which describes the evolution of the shoreline and beach slope as a function of the incident wave conditions. The model describes both the average beach profile changes (i.e., the shoreline position and the average beach slope) along an open beach as well as the shoreline changes associated with a single groin. Additional work is needed to incorporate wave diffraction effects.

4. There is a limit to the extent that vertically integrated energetics-based sediment transport models can be used to describe surfzone sediment transport processes. The present models can be modified by including sediment threshold of motion effects, a variable drag-coefficient, and a different formulation for the rate of energy dissipation in the surfzone. These modifications will probably result in an improvement to the models, especially in the case of the on-offshore sediment transport model. Most likely, however, significant progress in developing improved sediment transport models will only come through an increase in our understanding of the vertical structure of the sediment concentration and velocity fields.

Based on these findings, it is recommended that future work focus on measuring the vertical structure of the sediment concentration and fluid velocity fields. A parallel effort should focus on developing improved models for wave bore dynamics, surfzone boundary layer development, bedload, and suspended load mechanics. Improved models for the longshore and on-offshore currents in the surfzone are also needed.

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Table 1. Longshore Transport Data

| Source | No. OBS | W (cm/sec) | $u_{mb}/W$ | $\alpha_b$ (deg) | Kmeas |
|---|---|---|---|---|---|
| Laboratory | | | | | |
| Saville (1950) | 7 | 4.0 | 8.3 | 6 | 0.16 |
| Shay & Johnson | 3 | 4.0 | 8.5 | 6 | 0.22 |
| Field | | | | | |
| Bruno et al. (1980) | 7 | 2.5 | 113 | 0.2 | 0.87 |
| Dean et al. (1983) | 7 | 2.7 | 31 | 15 | 1.15 |
| Komar & Inman (1970) | | | | | |
| El Moreno | 10 | 9.5 | 11 | 9 | 0.82 |
| Silver Strand | 3 | 2.0 | 72 | 4 | 0.66 |
| Kraus et al. (1983) | | | | | |
| Ajiguara | 2 | 3.2 | 38 | 4 | 0.31 |
| Hirono | 2 | 9.5 | 16 | 2.5 | 0.11 |
| Shimokita | 1 | 2.7 | 48 | 6.0 | 0.61 |
| Moore & Cole (1960) | 1 | 20.5 | 9.3 | 12 | 0.19 |

Table 2. Estimated Wave Velocity Moment Coefficients for Equation 35

| Parameter | a | b | c | d |
|---|---|---|---|---|
| $u_m$ (m/sec) | 28.4 | -0.288 | 1.29 | 0.283 |
| $\delta_u$ | -6.50 | -0.0158 | -4.29 | 0.138 |
| $\Psi_1$ | 2.27 | -0.141 | -12.7 | 0.547 |
| $\Psi_2$ | 3.52 | -0.287 | -25.0 | 1.13 |

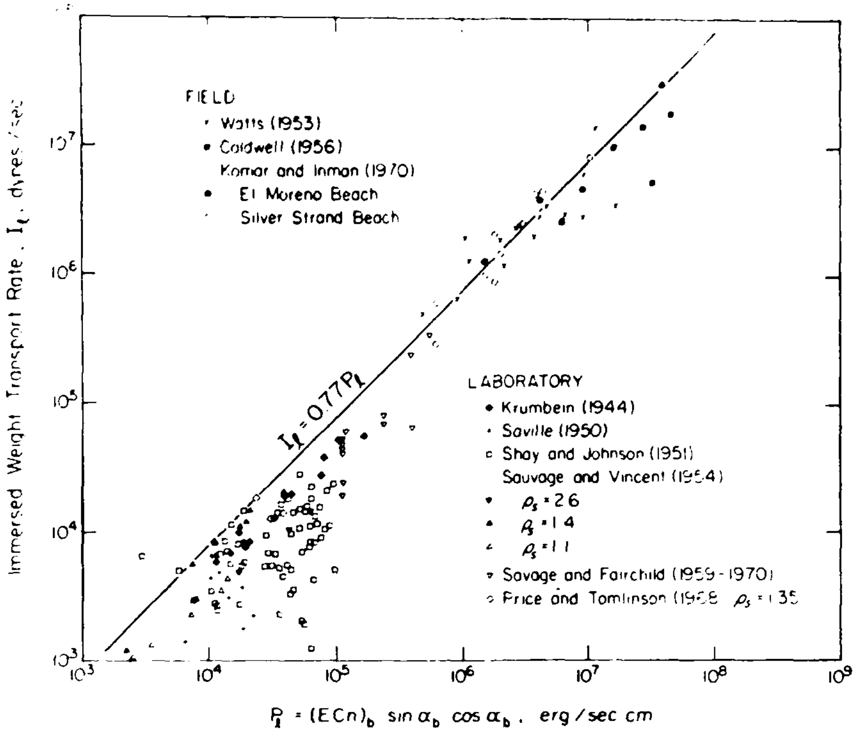


Figure 1. The wave power equation describes the field measurements of longshore transport relatively well, however, it over-predicts the laboratory measurements (from P.D. Komar and D.L. Inman (1970), "Longshore sand transport on beaches," Journal of Geophysical Research, vol 75, no. 30, 1970, Fig. 5, pg 5922).

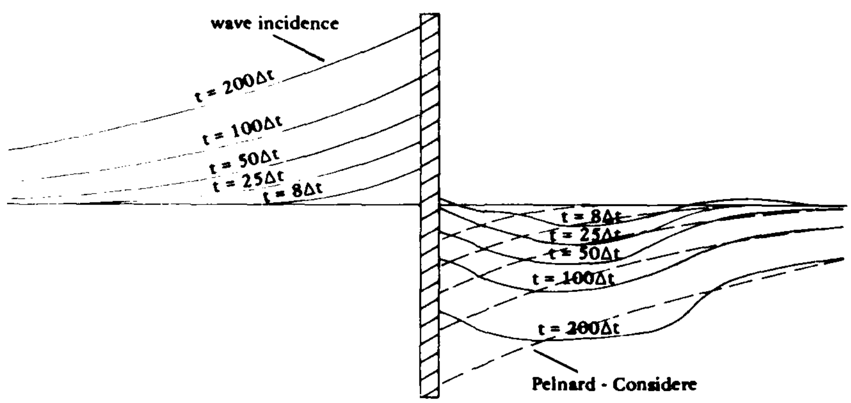


Figure 2. Progressive accretion and erosion near a groin, based on a one-line numerical model. The solid lines represent the solution with diffraction, the dashed lines without (from W.T. Bakker, K. Breteler, A. Roos, 1970, "The dynamics of a coast with a groyne system," ASCE Proc. 12th Coastal Eng. Conf., Fig. 5, pg 1004, vol 2).

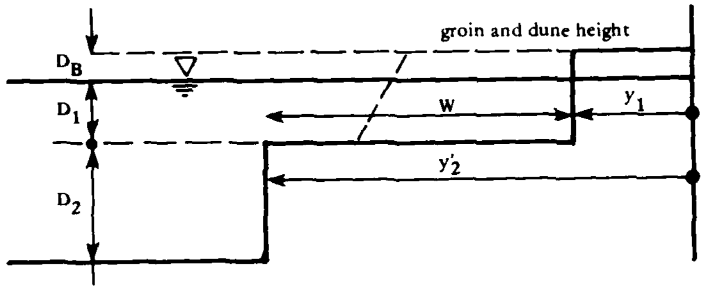


Figure 3. Definition sketch for two-line shoreline evolution model.

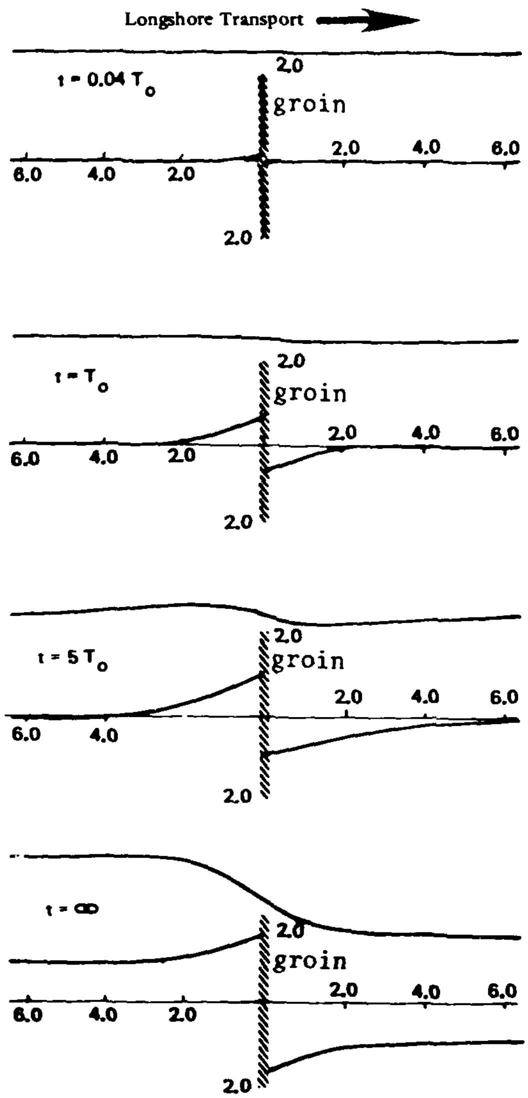


Figure 4. Evolution of the shoreline and offshore beach limit near a groin (from W.T. Bakker, 1968, "The dynamics of a coast with a groin system," ASCE Proc. 11th Coastal Eng. Conf., Fig. 8, pg 497, vol 1).

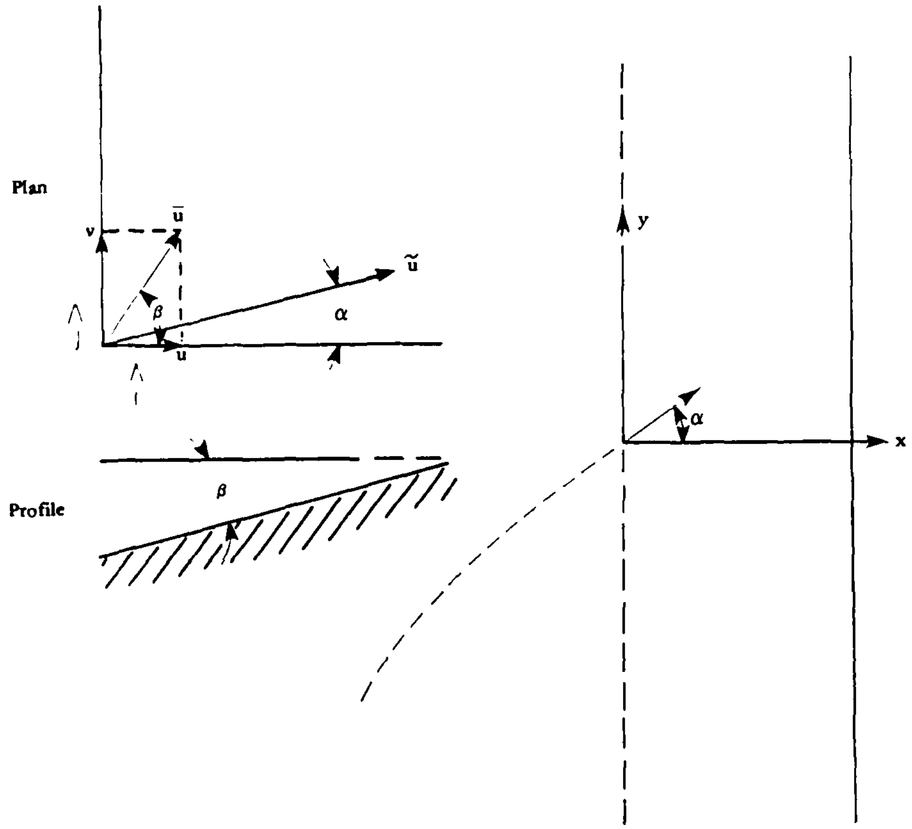


Figure 5. Schematic diagram showing the position of the beach (solid line), the breakpoint (dashed line), the wave angle $\alpha$ and the oscillatory and steady water velocities u and u.

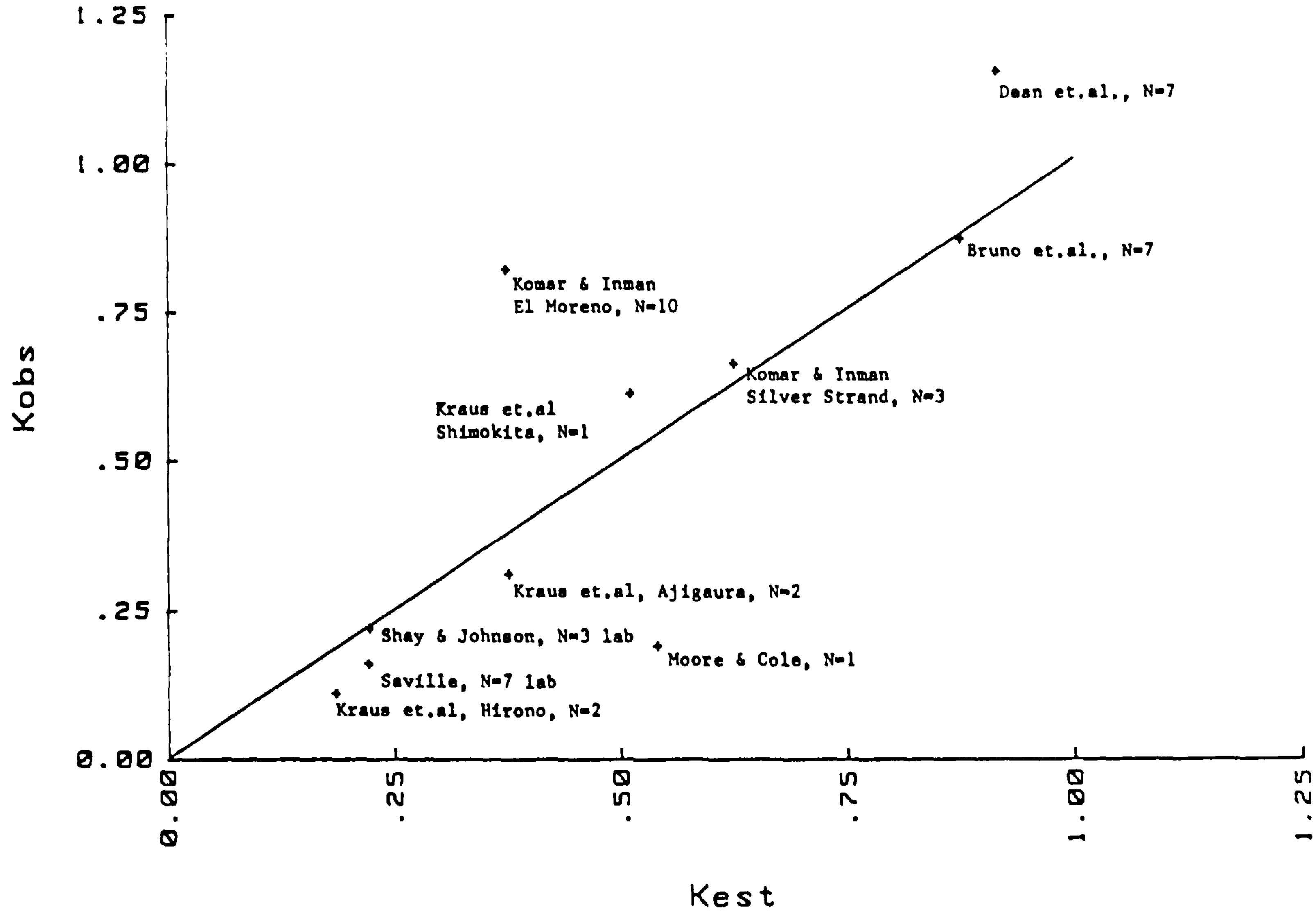


Figure 6. Observed values of the wave power coefficient versus estimated values using Equation 33. The solid line represents a one to one correspondence.

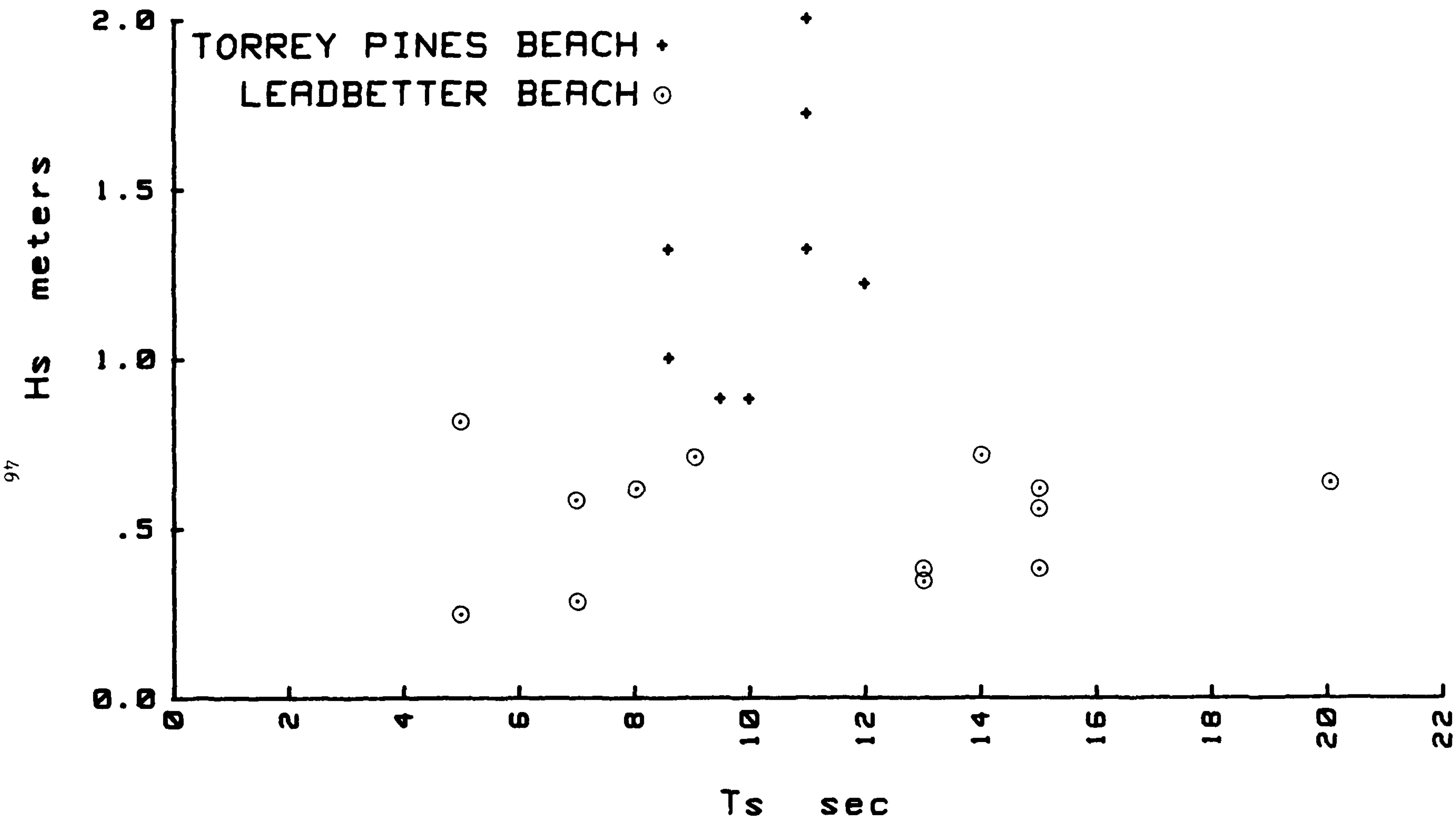


Figure 7. Significant wave height and wave period characteristics for the Torrey Pines Beach and Leadbetter Beach data sets.

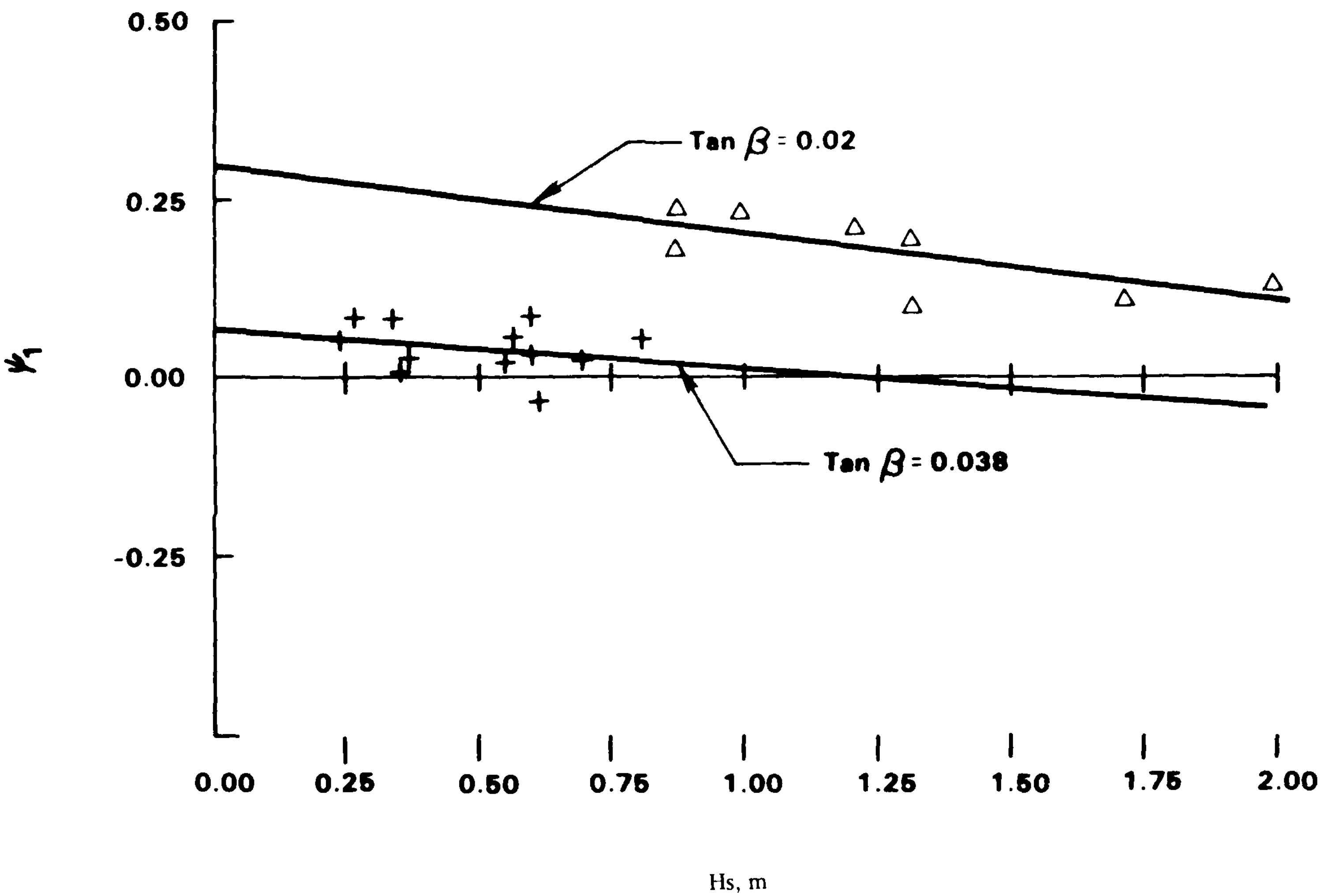


Figure 8. The spatially-averaged skewness parameter, $\psi_1$, was found to be a linear function of the incident significant wave height and the nearshore beach slope. The solid lines represent a least squares fit to the data.

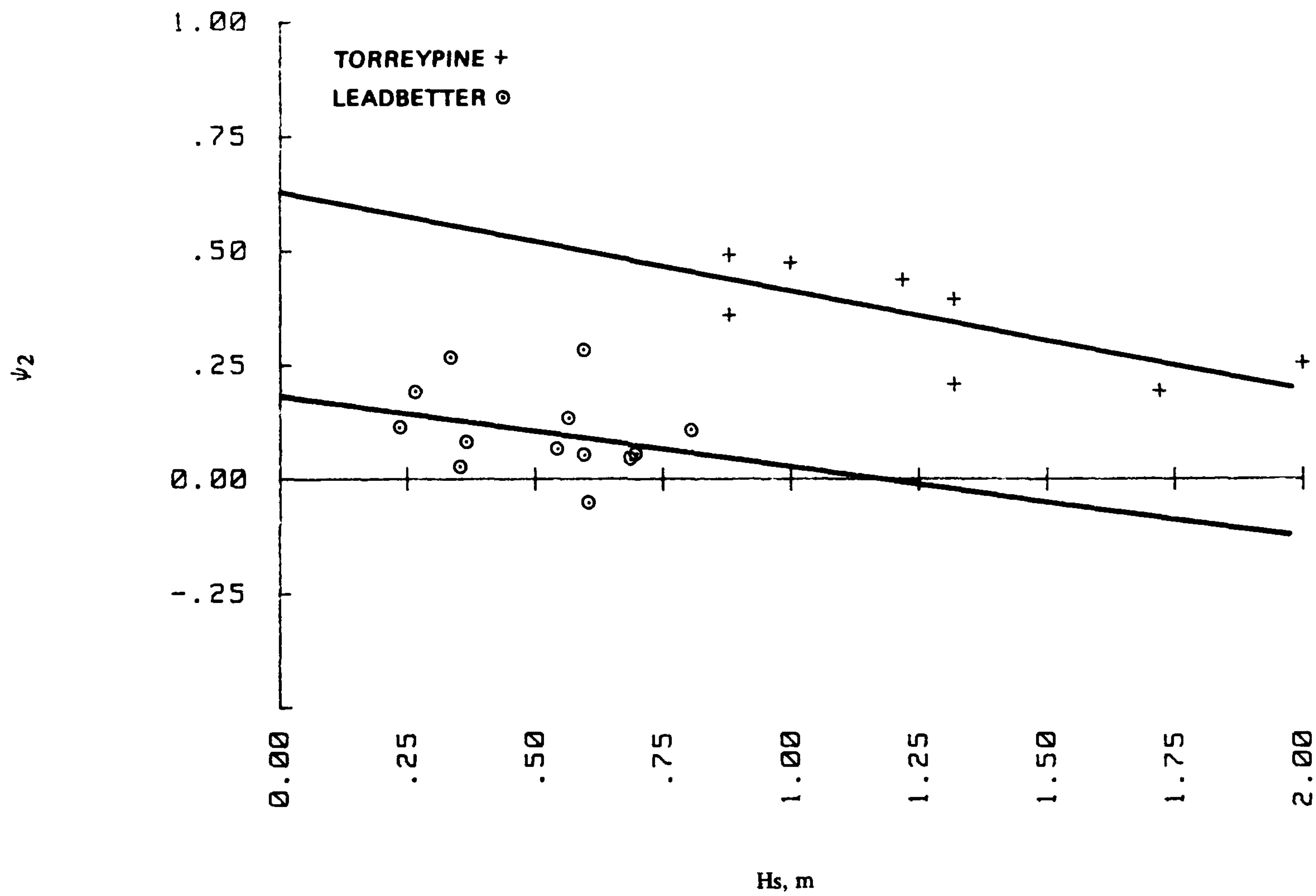


Figure 9 The spatially-averaged wave skewness parameter, $\psi_2$, was found to be linearly related to the incident significant wave height and the nearshore beach slope. The solid lines represent a least squares fit to the data.

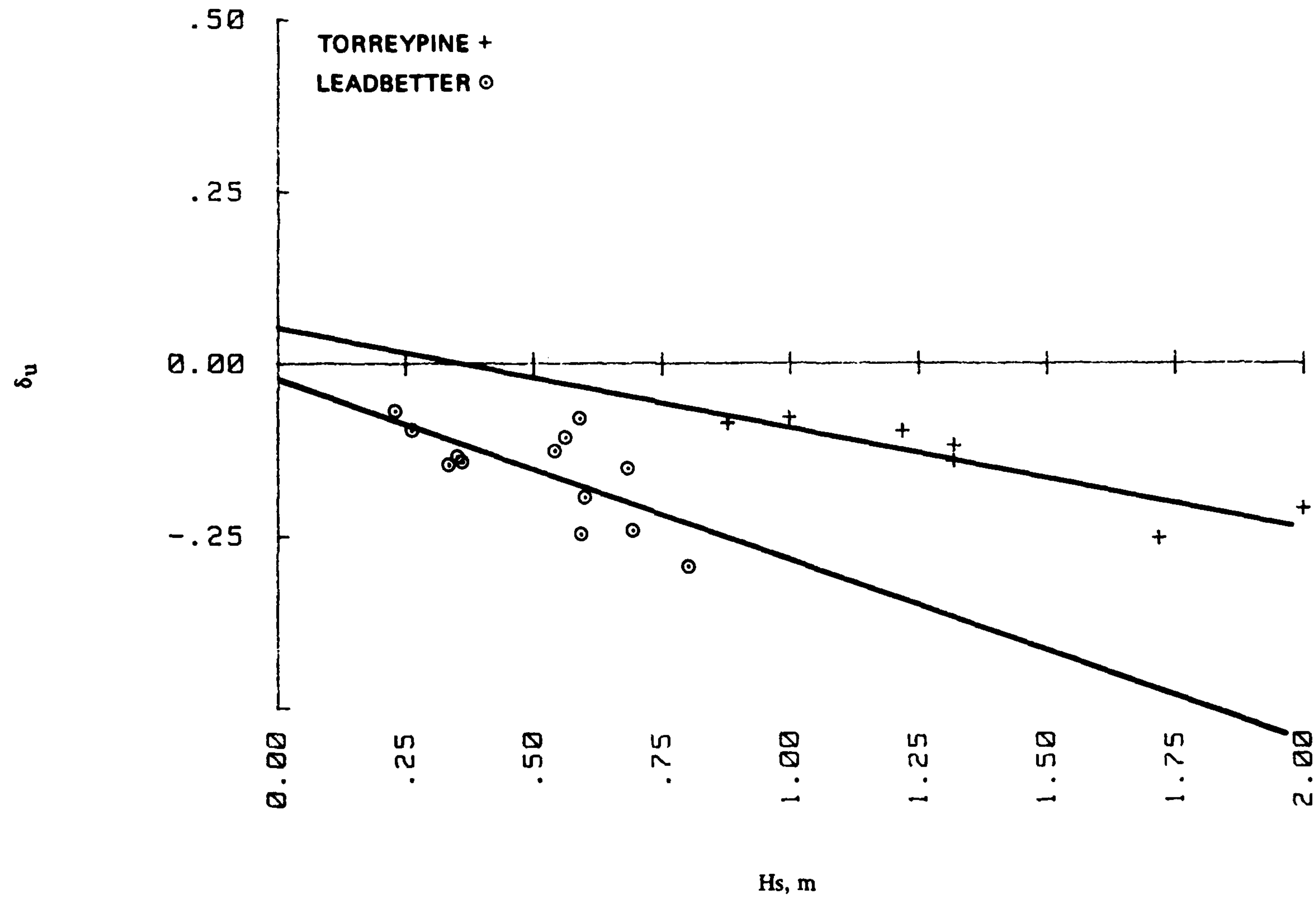


Figure 10. The spatially-averaged dimensionless mean onshore velocity, Su, was found to be linearly related to the incident significant wave height and the nearshore beach slope. The solid lines represent a least squares fit to the data.

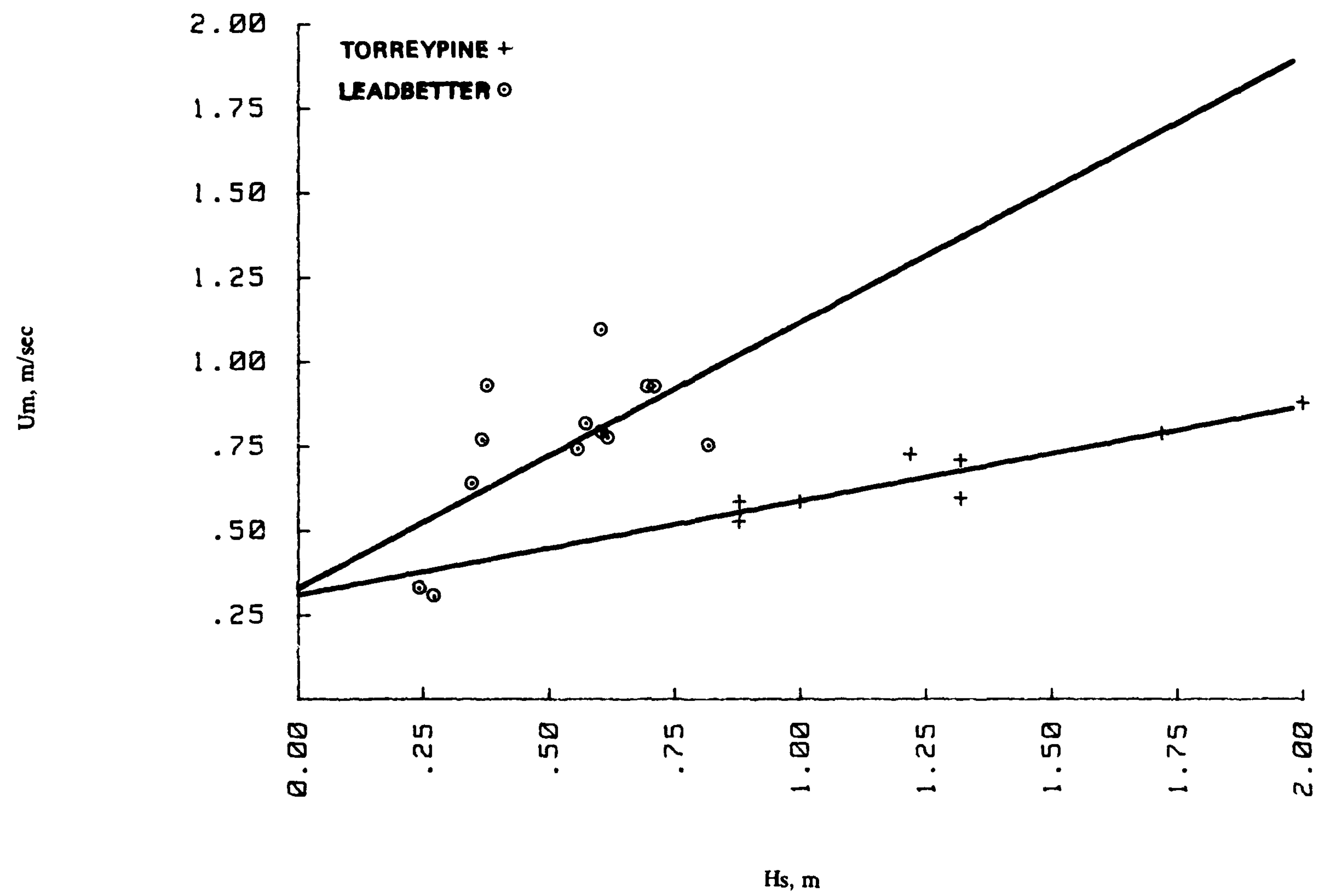


Figure 11. The spatially averaged orbital velocity magnitude, $u_m$, was found to be linearly related to the incident significant wave height. The solid lines represent a least squares fit to the data.

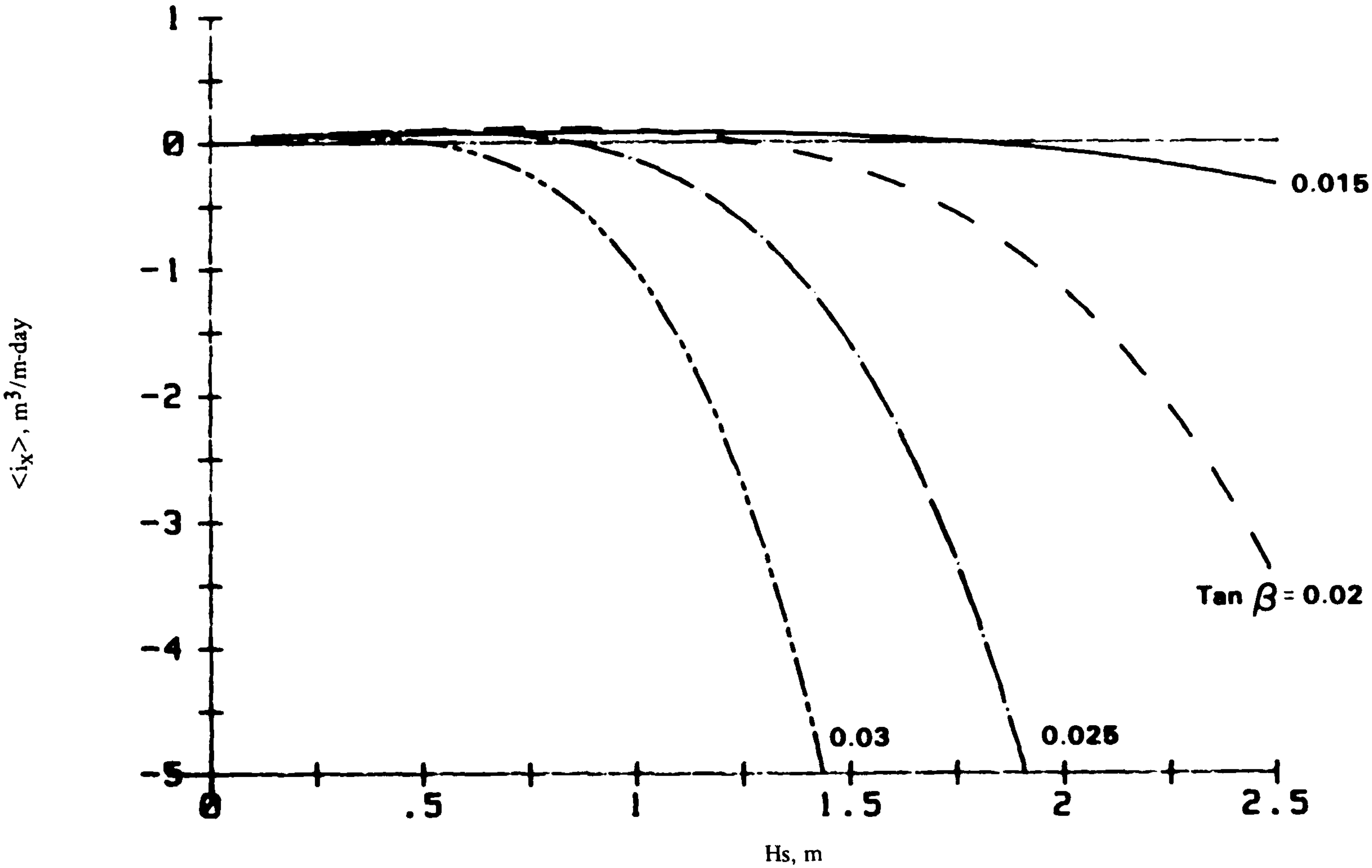


Figure 12. The predicted onshore transport rate is a function of the incident significant wave height and the nearshore beach slope. Note that small waves cause onshore transport, while large waves cause offshore transport.

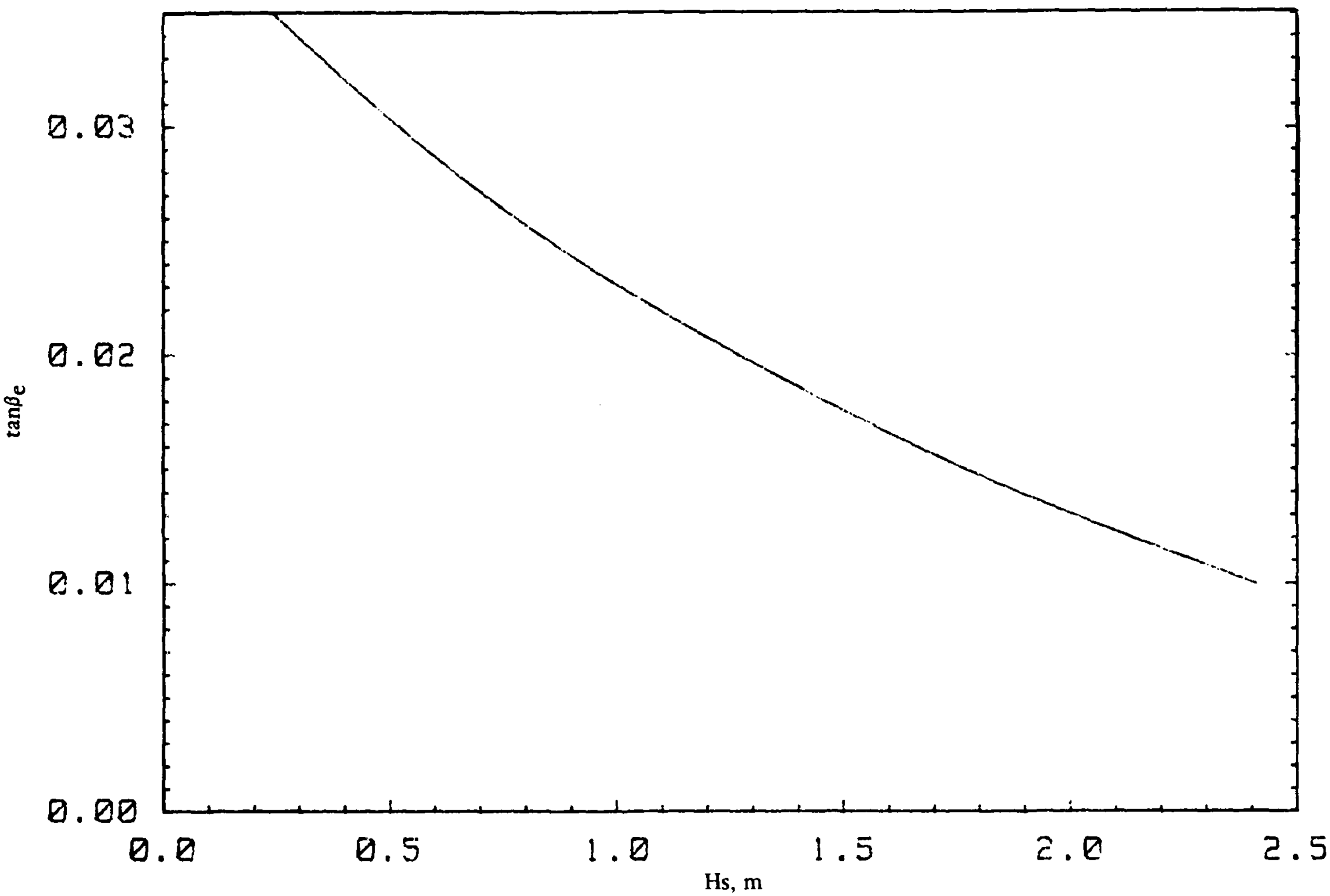


Figure 13. Predicted equilibrium beach slope as a function of the incident significant wave height. Note that the equilibrium beach slope decreases with increasing wave height.

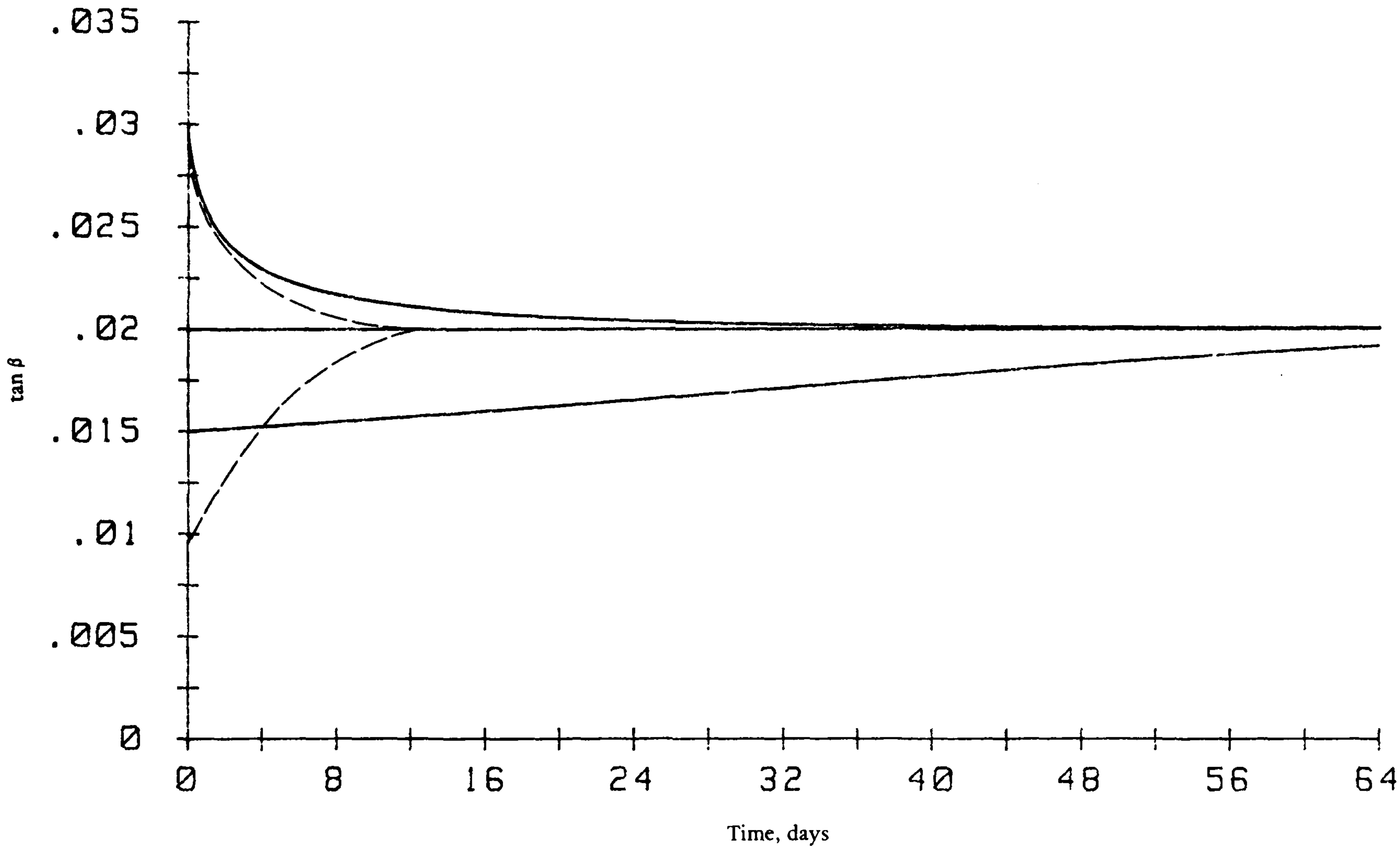


Figure 14. Comparison between the time evolution of the mean beach slope as predicted by the present on-offshore transport model (solid line) and the Bakker/Swart model (dashed line). The equilibrium beach slope is 0.02 while the initial beach slopes are 0.015 and 0.03.

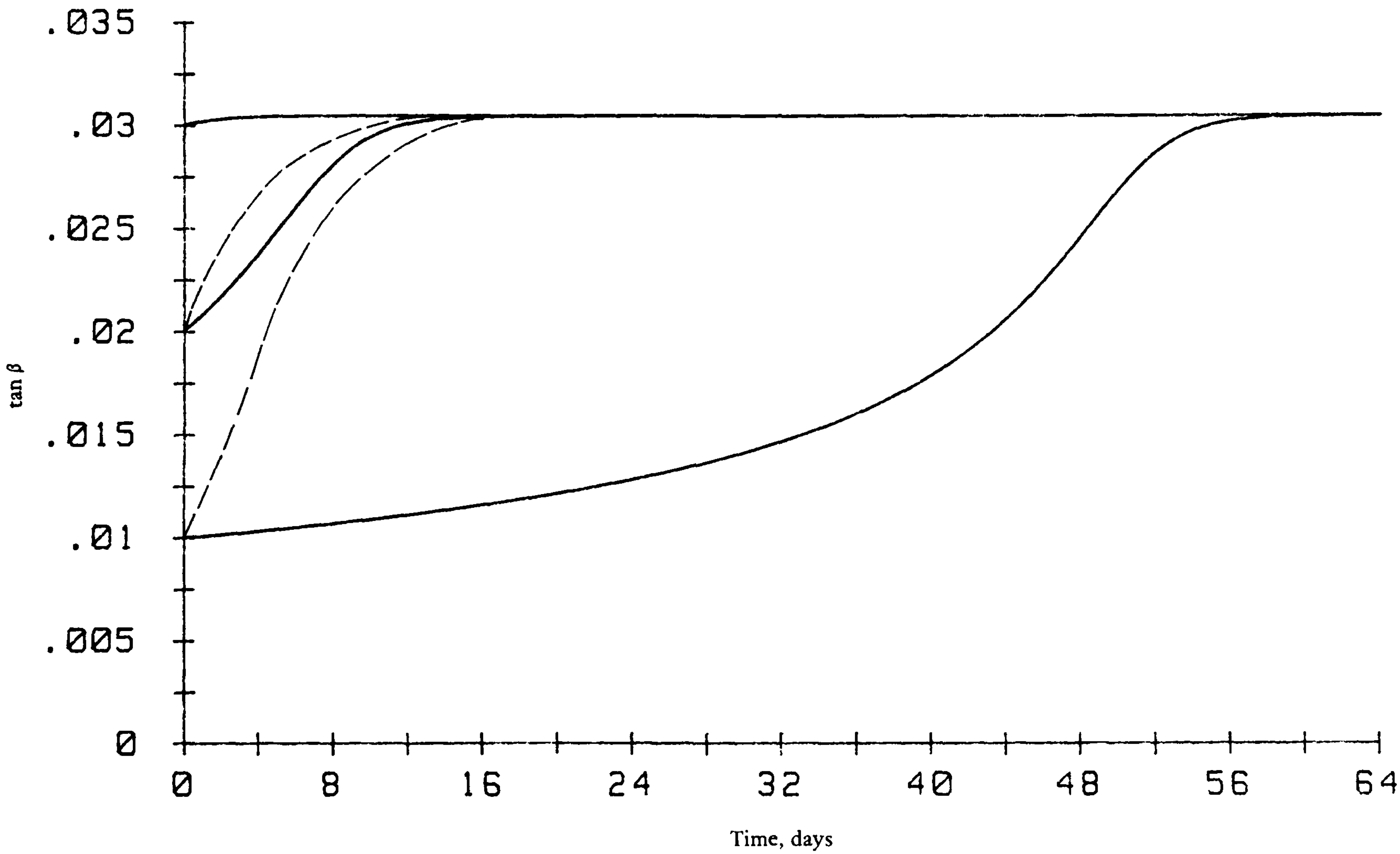


Figure 15. Comparison between the time evolution of the mean beach slope as predicted by the present on-offshore transport model (solid line) and the model by Bakker/Swart (dashed line). The equilibrium beach slope is 0.03 while the initial beach slopes are 0.01 and 0.03.

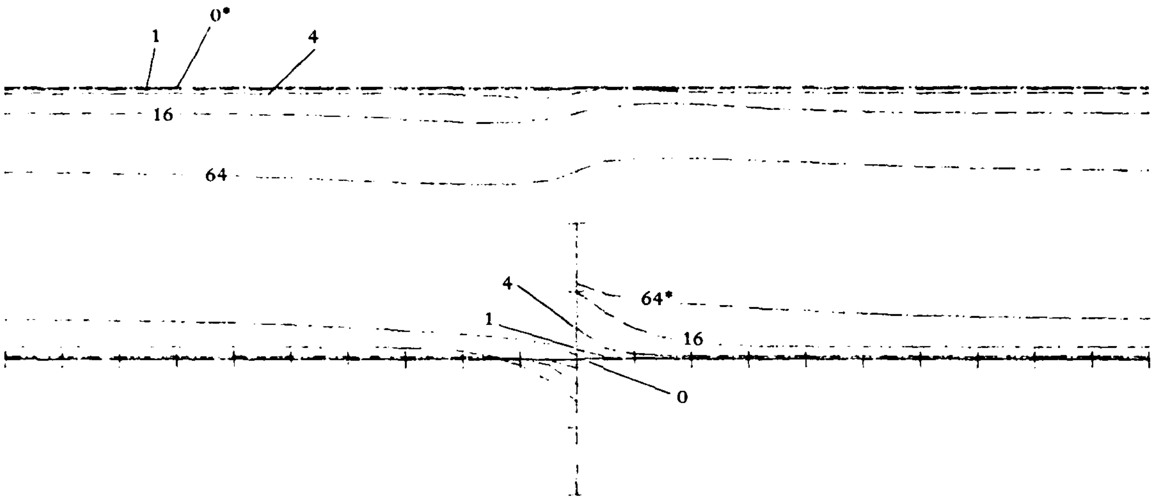


Figure 16. Time evolution of the shoreline around a single groin. The initial beach slope is steeper than the equilibrium slope, causing the overall beach to flatten while adjusting to the presence of the groin.

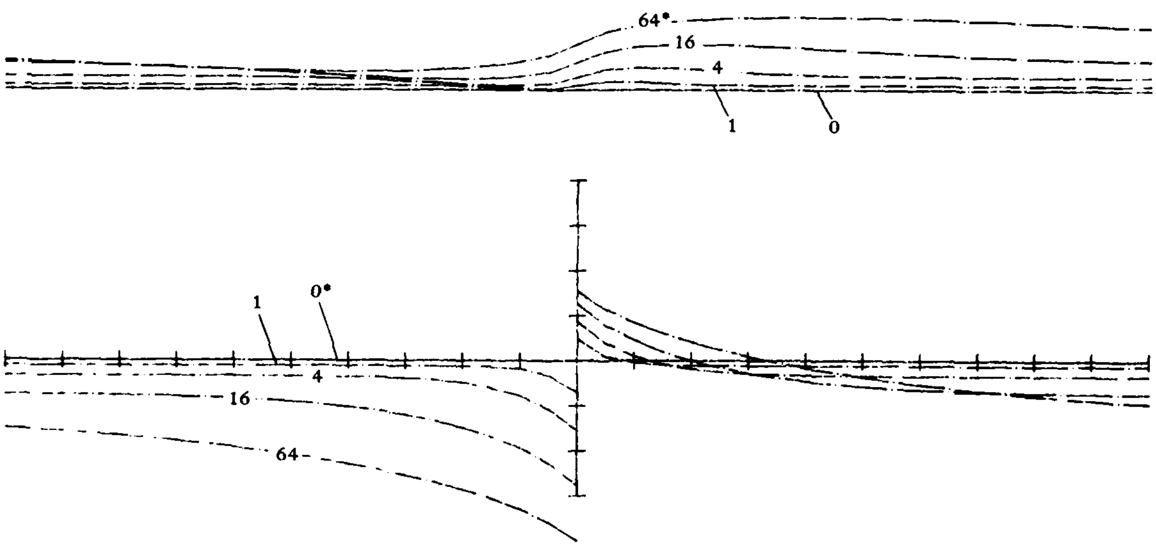


Figure 17. The time evolution of the shoreline around a single groin. The initial beach slope was flatter than the equilibrium slope causing the overall beach to steepen while adjusting to the presence of the groin.

* numbers = days

## Appendix A

## A DERIVATION OF AN EXPRESSION FOR THE BEACH SLOPE EXPRESSED AS A FUNCTION OF TIME, BASED ON THE BAKKER/SWART ON-OFFSHORE TRANSPORT MODEL

Assuming zero divergence of the longshore sediment transport rate, the two-line sand conservation equations (Equations 6 and 7) become

$$\left(D_1 + D_B\right) \frac{\partial x_1}{\partial t} = - q_x \tag{A-1}$$

$$D_2 \frac{\partial x_2}{\partial t} = q_x \tag{A-2}$$

where $q_x$, the on-offshore sediment transport rate is defined as

$$q_x = \text{Coff} \left(x_1 - x_2 + w\right) \tag{A-3}$$

and $w$ = the equilibrium value of $(x_2 - x_1)$. Defining

$$z = x_1 - x_2 + w \tag{A-4}$$

and taking the derivative of z with respect to t, the following result is obtained

$$\frac{1}{\text{Coff}} \frac{\partial q_x}{\partial t} = - q_x \left(\frac{1}{D_1 + D_B} + \frac{1}{D_2}\right) \tag{A-5}$$

where use has been made of Equations A-1 and A-2. Rearranging Equation A-4, and integrating, the following equation for the on-offshore transport rate is obtained

$$q_x = C\ e^{-Bt} \tag{-A6}$$

where C is an integration constant, and $B = \frac{Coff}{D_1 + D_B} + \frac{Coff}{D_2}$.

Applying the following initial condition (i.e., at time t = 0,)

$$q_x = Coff\left(\frac{-2\ D_1}{\tan\ \beta_i} + \frac{2\ D_1}{\tan\ \beta_e}\right) \tag{A-7}$$

where tan β is the initial beach slope, and tan β is the equilibrium beach slope, the constant C becomes

$$C = Coff\left(\frac{-2\ D_1}{\tan\ \beta_i} + \frac{2\ D_1}{\tan\ \beta_e}\right) \tag{A-8}$$

Substituting Equation A-7 into Equation A-5, and rearranging, the desired equation for the beach slope, tan β, is obtained

$$\tan\ \beta = \frac{\tan\ \beta_e}{\left(1 - e^{-Bt}\right) + \frac{\tan\ \beta_e}{\tan\ \beta_i} e^{-Bt}} \tag{A-9}$$

Appendix B

## A DERIVATION OF AN EQUATION FOR $\eta_1$

In a two-line model is it necessary to split the longshore transport rate into two parts, that which occurs shoreward of the $D_1$ depth contour, and that which occurs seaward of the $D_1$ depth contour. The parameter, $\eta_1$, is defined as the fraction of the total longshore transport rate occurring between the shoreline and the $D_1$ contour line. The normalized x coordinate position of this contour line $x_1^*$ is defined as

$$x_1^* = \frac{D_1}{h_b} \tag{B-1}$$

where: $h_b$ = water depth at the break point

In this way, $\eta_1$ can be expressed as

$$\eta_1 = \frac{\int_o^{x_1^*} \langle i_y \rangle dx^*}{\int_o^{\infty} \langle i_y \rangle dx^*} \tag{B-2}$$

Recalling that Equation 22, which defines the cross-shore distribution of the longshore sediment transport rate, $\langle i_y \rangle$, and assuming that the transport is primarily by suspension, then following a lengthy integration, the following expression is obtained

If $x_1^* \leq 1$ then

$$\eta_1 = \frac{\left(\frac{1}{C_3 + 5/2}\right)\left(\frac{1 - C_2}{C_2 - C_3}\right) x_1^{*\,C_2+5/2} + 2/7\ x_1^{*\,7/2}}{\frac{1}{C_3 + 5/2} + \frac{2}{7} - \left(\frac{1}{C_2 - 5/4}\right)\left(\frac{1 - C_3}{C_2 - C_3}\right)} \qquad \text{(B-3)}$$

If $x_1^* > 1$ then

$$\eta_1 = 1 + \frac{C_1}{C_2 - 5/4}\left(\frac{1 - C_3}{C_2 - C_3}\right) x_1^{C_2 - 5/4}$$

Finally, if P = 0.2 then

$$\eta_1 = -1.69\ x_1^{*\,4.11} + 2.34\ x_1^{*\,3.5} \qquad x_1^* \leq 1$$

$$\eta_1 = 1 - 0.35\ x_1^{*\,-3.61} \qquad x_1^* > 1 \qquad \text{(B-4)}$$

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AFESC AFESC TST, Tyndall AFB, FL; HQ AFESC/TST, Tyndall AFB, FL
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ARMY 36th Engineer Group AFVK-C (Capt P Topp), Ft Benning, GA; AMCSM-WCS, Alexandria, VA; ARRADCOM, Dover, NJ; BMDSC-RE (H. McClellan) Huntsville AL; DAEN-MPE-D Washington DC; DAEN-MPU, Washington DC; POJED-O, Okinawa, Japan; QTRMSTR Scol (Code ATSM-CD) Ft. Lee, VA; R&D Ctr, STRNC-US (J Siegel), Natick, MA
ARMY - CERL Library, Champaign IL; Spec Assist for MILCON, Champaign, IL
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ARMY ENGR DIST. Library, Portland OR
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ARMY MTMC Trans Engr Agency MTT-CE, Newport News, VA
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DIA DB-6E1, Washington, DC; DB-6E2, Washington, DC
DLSIE Army Logistics Mgt Center, Fort Lee, VA
DNA STTI/TL, Washington, DC
DOE Wind/Ocean Tech Div, Washington, DC
DTIC Alexandria, VA
DTNSRDC Code 1175, Annapolis, MD; Code 119, Annapolis, MD; Code 1250, Annapolis, MD; Code 1561, Bethesda, MD; Code 1568, Annapolis, MD; Code 1706, Bethesda MD; Code 172, Bethesda, MD; Code 284, Annapolis, MD; Code 2842, Annapolis, MD; Code 4111 (R. Gierich), Bethesda MD; Code 4120, Annapolis, MD
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NAVCOMMSTA Code 401, Nea Makri, Greece; Dir, Maint Control, PWD, Diego Garcia; PWO, Exmouth, Australia
NAVCONSTRACEN Code 00U15, Port Hueneme CA; Code B-1, Port Hueneme, CA; Curriculum/Instr. Stds Offr, Gulfport MS
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NAVFACENGCOM CO, Alexandria, VA; Code 03, Alexandria, VA; Code 03T (Essoglou), Alexandria, VA; Code 04A1, Alexandria, VA; Code 04B3, Alexandria, VA; Code 04M, Alexandria, VA; Code 04T1B (Bloom), Alexandria, VA; Code 04T4, Alexandria, VA; Code 04T5, Alexandria, VA; Code 06, Alexandria VA; Code 07A (Herrmann), Alexandria, VA; Code 09M124 (Tech Lib), Alexandria, VA; Code 100, Alexandria, VA; Code 1113, Alexandria, VA

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NAVMAG SCE, Subic Bay RP
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NAVPETOFF Code 8D107, Alexandria, VA
NAVPETRES Director, Washington DC
NAVPGSCOL C. Morers, Monterey, CA; Code 1424, Library, Monterey, CA; Code 61WL (O. Wilson), Monterey, CA; Code 68 (C.S. Wu), Monterey, CA; E. Thornton, Monterey, CA
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NAVSEASYSCOM Code 035, Washington DC; Code 05G13, Washington, DC; Code 05R12, Prog Mgr Washington, DC; Code 06H4, Washington, DC; Code C132, Washington, DC; Code PMS-396.3211 (J. Rekas) Washington, DC
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NAVSECGRUCOM Code G43, Washington, DC
NAVSHIPYD Carr Inlet Acoustic Range, Bremerton, WA; Code 202.5 (Library), Bremerton, WA; Code 280, Mare Is., Vallejo, CA; Code 280.28 (Goodwin), Vallejo, CA; Code 380, Portsmouth, VA; Code 410, Mare Is., Vallejo CA; Code 440, Bremerton, WA; Code 440, Bremerton, WA; Code 440, Norfolk, VA; Code 440, Portsmouth, NH; Code 440.4, Bremerton, WA; Dir, Maint Control, PWD, Long Beach, CA; PWO, Charleston, SC; PWO, Mare Island, Vallejo, CA
NAVSTA A. Sugihara, Pearl Harbor, HI; CO, Long Beach, CA; CO, Roosevelt Roads, PR; Dir Mech Engr 37WC93 Norfolk, VA; Dir, Engr Div, PWD (Code 18200), Mayport, FL; Engrg Dir, Rota, Spain; PWO, Mayport, FL; SCE, Subic Bay, RP; Util Engrg Offr, Rota, Spain
NAVSUPPFAC Dir, Maint Control Div, PWD, Thurmont, MD
NAVSUPPO Security Offr, La Maddalena, Italy
NAVSURFWPNCEN Code E211 (C. Rouse), Dahlgren, VA; DET, Security Offr, Silver Springs, MD; G-52 (Duncan) Dahlgren, VA; PWO, Dahlgren, VA
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NAVWARCOL Fac Coord (Code 24), Newport, RI
NAVWPNCEN Code 2636, China Lake, CA; DROICC (Code 702), China Lake, CA
NAVWPNSTA Dir, Maint Control, PWD, Concord, CA; Engrg Div, PWD, Yorktown, VA; PWO, Charleston, SC; PWO, Code 09B, Colts Neck, NJ; PWO, Seal Beach, CA
NAVWPNSTA PWO, Yorktown, VA
NAVWPNSUPPCEN Code 09 Crane IN
NETC PWO, Newport, RI
COMEODGRU OIC, Norfolk VA
NCR 20, CO, Gulfport, MS; 20, Code R70
NMCB 74, CO; FIVE, Operations Dept; Forty, CO; THREE, Operations Off.
NOAA (Mr. Joseph Vadus) Rockville, MD; Library, Rockville, MD
NORDA Code 410, Bay St. Louis, MS; Head, Geotech Br (Code 363), Bay St. Louis, MS; Ocean Rsch Off (Code 440), Bay St. Louis, MS
NRL Code 5800 Washington, DC; Code 5843 (F. Rosenthal) Washington, DC; Code 8441 (R.A. Skop), Washington, DC

USCG Code 2511 (Civil Engrg), Washington, DC
NSC Cheatham Annex, PWO, Williamsburg, VA; Code 54.1, Norfolk, VA; Code 700 Norfolk, VA; SCE, Charleston, SC; SCE, Norfolk, VA; Security Offr (Code 44), Oakland, CA
NSD SCE, Subic Bay, RP
CBU 401, OICC, Great Lakes, IL
NUCLEAR REGULATORY COMMISSION T.C. Johnson, Washington, DC
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ONR Code 481, Bay St. Louis, MS; DET, Dir, Boston, MA
OCNR Code 421 (Code E.A. Silva), Arlington, VA; Code 700F, Arlington, VA
PERRY OCEAN ENG R. Pellen, Riviera Beach, FL
PHIBCB 1, CO San Diego, CA; 1, ELCAS Offcr, San Diego, Ca; 1, P&E, San Diego, CA; 2, Co, Norfolk, VA
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